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Q Street Bridge at Washington

The valley of Rock Creek, separating from the main part of the city that section of Washington, D. C., which was at one time the old city of Georgetown, is a deep winding gorge through which a small stream flows to the Potomac. The increasing growth of the city toward the north and the west has made necessary within the past ten to fifteen years the construction of a number of high-level bridges across the gorge. Four of these bridges are of the arch type—the old Massachusetts Ave. viaduct, a brick, small-opening arch with a heavy fill, more like a culvert than a bridge; the Connecticut Ave. bridge, finished about ten years ago, well known as one of the most beautiful bridges in the country; the Pennsylvania Ave. bridge, which incloses the old water-main arch of the Washington aqueduct and which was described in *Engineering News* of Dec. 16, 1915; and finally the Q St. bridge, which is the subject of this article.

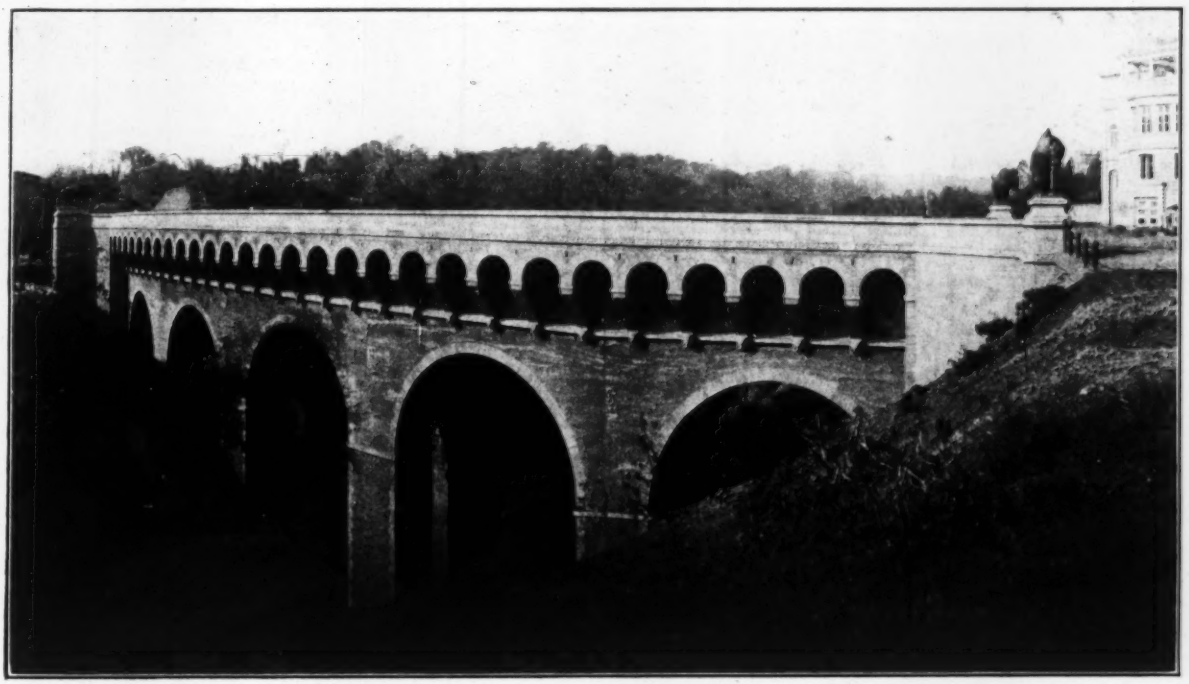


FIG. 1. THE NEW Q ST. BRIDGE ACROSS ROCK CREEK, WASHINGTON, D. C.

The Q St. bridge is noteworthy because of the special effort that was made in its design to have it conform architecturally with the very high-class residences that are going up around the east end of the bridge, and to fit in with the proposed park system that is expected soon to reach down the valley from the famous Rock Creek Park beyond. Unfortunately at present the west end of the bridge and the valley itself are in a decidedly unkempt condition, not at all in keeping with the other end of the bridge or the bridge itself. However, this state of affairs will probably soon be changed.

In the desire to achieve a desirable architectural effect the city officials called in a consulting architect in the person of Glenn Brown, who was formerly secretary of the American Institute of Architects, and it is to Mr. Brown and to the higher officials in the District government rather than to the city engineers that the decorative features of the bridge are due. These effects are evident not only in the decoration, adornment and color of the bridge, but also in the fact that it is built on a curve in plan, although there seems to be no very good structural reason why this extremely difficult and annoying type of construction should have been adopted. It is understood, however, that someone in higher authority had been particularly impressed with the artistic effect of the famous old curved arch viaduct of the Baltimore & Ohio R.R. at Relay, Md., just outside of Baltimore, and took occasion to use the same general scheme for the Q St. bridge. The plan is partly allowable because the approach streets on the opposite sides of the gorge are not in alignment.

The bridge for the time being will be used only as a highway crossing. It consists of five arch spans varying

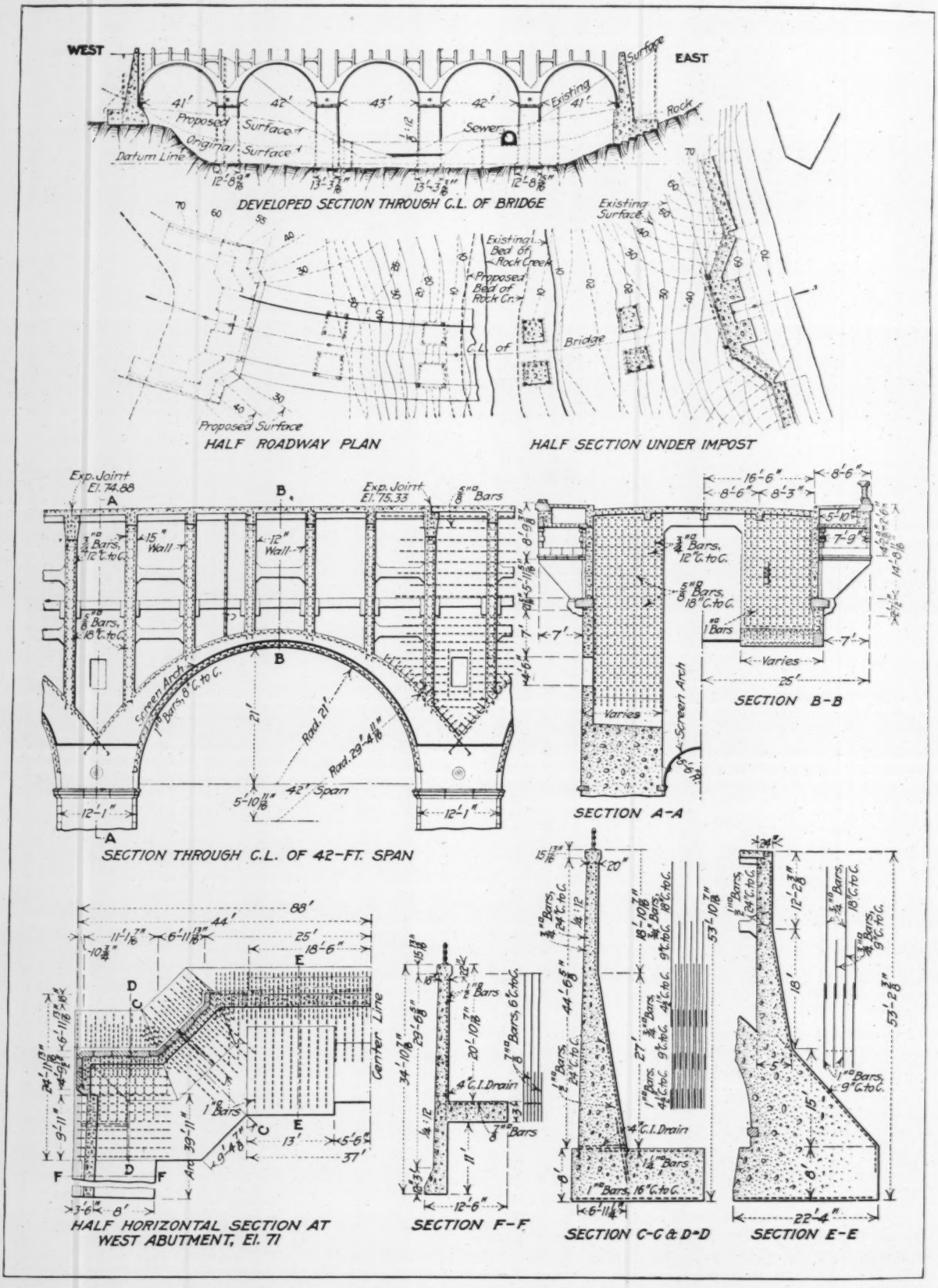


FIG. 2. DETAILS OF THE NEW Q ST. BRIDGE ACROSS ROCK CREEK GORGE, WASHINGTON, D. C.

from 43 ft. clear span in the center to 42 ft. for the next adjoining two, and 41 ft. for the end spans. It carries two 7-ft. sidewalks and a 33-ft. roadway. There are twin arch ribs, each of solid reinforced section footing into separated piers. Screen arches of thin section span between the main arches to form a solid soffit.

The reinforced-concrete floor slabs are carried on two-way beams that rest on transverse walls reaching down to the arch ribs. These walls are cross-braced longitudinally, as shown in Fig. 2. Solid spandrel walls form a showing face, as may be noted from the view in Fig. 1. Sidewalks are carried on overhanging beams that are joined with arches in a highly ornamental fashion, as noted in the view of the whole arch.

The center line of the roadway is curved to a radius of 474 ft. The piers are curved on their outer face and dimensioned to fit the radii, as shown in the plan Fig. 2. The arch ribs themselves, as well as the spandrel walls, are also bent to this same curvature. In construction this did not cause as much difficulty as might be imagined, because the arches are so short that the offset was in no case very large. The curve is carried to the roadway and shows in the curbstone and in the railing. It is so slight that a casual observer on the bank, so far away as not to be able to see the roadway with its curbs and railing, might not realize that the bridge is curved in plan unless the eye took in the line of the street continuing the bridge on the far side. In the first-page view this street may be noted as the white strip extending to the right at the far side of the bridge.

Details of Abutment Design

The abutments are of the wing type and somewhat unusual in design. Fig. 2 shows some of the details of the west abutment. The other is quite similar. Both abutments and the piers foot on rock. As will be noted in the drawing, the wings are of complex plan, but the general type of construction is a thin retaining wall backed with earth. At the arch rib the wall is extended backward to take care of the thrust from the arch, but the middle section, in Fig. 2, is typical of the other parts of the wall. This wall is 46 ft. high to the top of the solid 8-ft. concrete base. It is 1 ft. 8 in. thick down to 18 ft. below the top and increases uniformly in thickness from that point to the top of the base wall, where it has a thickness of 7 ft., reinforced as shown in the section.

At the rear of the wing wall, where the slope of the bank began to come up so high on the wall that it was not thought worth while to excavate clear to the bottom, the wall itself was carried down to a depth required by the showing face. The fill was made on top of a relieving wall 3 ft. thick, which rested on the existing earth and which was joined by steel and concrete to form an integral part of the main wall. In this way the thrust on the showing wall was considerably reduced. The faces of all of the walls are slightly battered.

It might be noted that in view of a disastrous experience some ten years ago in the Connecticut Ave. bridge in Washington, the engineers of the city made special provision that the abutment between the outer lines of the bridge should not be completely back-filled until after the arch centers were struck.

The decorations and ornaments to the bridge are a special feature. The tone of the bridge is red, the main part of the spandrel walls being of a reddish concrete bush-hammered for rough surface. The quoins, coping, railing and abutment faces are of a red sandstone. The specifications required that the concrete face should be made of an aggregate formed from the crushed sandstone which went into the cut stone of the bridge, but when a test concrete was made in this way it was found that the cement and sand so nullified the reddish tint of the crushed sandstone as to give a concrete in no way resembling in tone the original sandstone. After considerable experiment and search, however, a red sand was found in a pit in Virginia near Washington which, when mixed with cement, formed a mortar very similar in tone and color to the sandstone. Unfortunately the effect of the bushhammered concrete is somewhat lessened by

FIG. 3. ONE OF THE BRONZE BUFFALOES ON THE Q ST. BRIDGE
A. Phimister Proctor, sculptor

streaks in the concrete face where patches of different-colored concrete got against the form. This may be noticed in the view in the first page. It may disappear in time when the concrete surface has weathered somewhat.

In addition to the ordinary bridge decorations in the form of cut-stone quoins and coping, the Q St. bridge has also very ornamental carving of the railing and the carved "Aztec's head" at the base of each cantilever. In addition the center of each spandrel wall between arches has a highly ornamental monogram D. C., for the District of Columbia. Another innovation is the placing of the lamp posts on "isles of safety" on the center line of the roadway.

The crowning decorations are the four huge bronze buffaloes which face each other on the pedestals at each approach. These buffaloes were modeled by A. Phimister Proctor, of New York, and cost the City of Washington about $10,000. They make fitting companion animals to the lionesses of the Piney Branch bridge and the lions of the Connecticut Ave. bridge of the same city. A view of one of these bronzes is shown in Fig. 3.

The Q St. bridge was built by A. L. Guidone & Co., of New York City, under the general direction of D. E. McComb, Engineer of Bridges of the District of Columbia. Glenn Brown, of Washington, D. C., as stated above, was consulting architect.

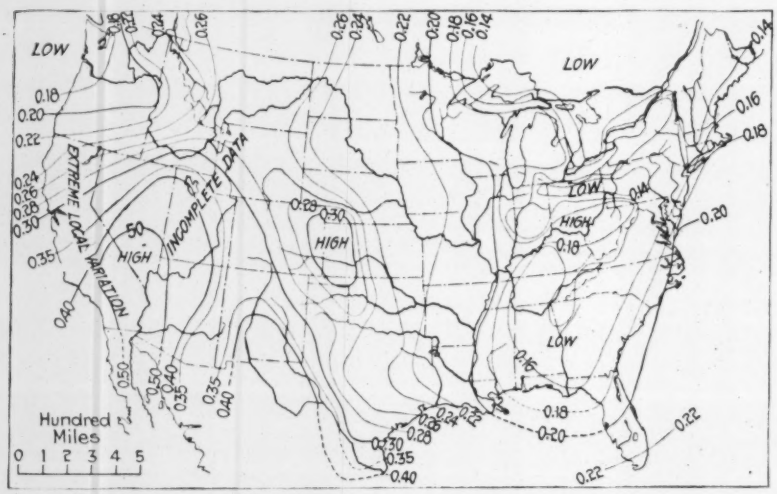
Variation in Annual Rainfall

By Allen Hazen*

Good maps showing the normal annual precipitation throughout the United States are published by the Weather Bureau and by the United States Geological Survey. Numerous maps of states show normal annual rainfall with more precision, because a smaller interval between the lines can be used with the larger scale. With the aid of these maps and the full records of the Weather Bureau, which can be obtained from the Government, it is possible to get quickly a fair idea of the mean annual rainfall conditions at any point in the United States, as far as shown by existing data.

The variation in rainfall is a matter of almost as much practical importance as its amount. The seasonal variations may be found from certain Weather Bureau tables that show the normal precipitations by months, and also from certain maps showing seasonal fluctuations. The annual variations are equally important, but are not as easily ascertained. In hydraulic literature their treatment has been inadequate.

One of the most noteworthy investigations of this subject is that of Sir Alexander Binnie, presented many years ago to the Institution of Civil Engineers[1] and more recently without essential change in his book on water-supply[2].



COEFFICIENT OF VARIATION OF ANNUAL RAINFALL IN THE UNITED STATES

During the past year the writer has made an effort to ascertain what may be expected in the way of variation in annual rainfall in the United States. This work has been done to aid in understanding rainfall and runoff data.

Measures of Variation

There are many possible measures of variation. The one selected as most suitable for this study is the "coefficient of variation" as defined in works on statistics. It is easily obtained by calculation from the official records and seems better adapted to the service than any other that has been suggested. The coefficient of variation is determined by the following simple procedure:

The annual rainfalls for each year of the whole record period are obtained. The mean of this series is usually published by the Weather Bureau at the bottom of the column. The difference between the actual rainfall for any year and the mean of the whole record period is the variation (from the mean) for that year. These variations are found for each year of the series. Each one is then squared. The mean of these squares is then found, and its square root is taken. This is the standard variation. The coefficient of variation is the ratio of the standard variation to the mean. A few short-cuts are possible, and after a little practice the coefficient can be found from a 40-yr. record in about 40 min.

Published records of rainfall were obtained from the Weather Bureau. They frequently covered the records only up to 1908 or 1909. The more recent records are available in other publications, but to use them involved new listing and computation of the means. For this purpose the additional listing and averaging needed to make them available were not undertaken. Not all the records were used. Many of them were too short to be of much service. In general the longest and apparently best records, scattered as evenly as possible all over the country, were selected to the number of several hundred. The coefficients for each series were determined and plotted on an outline map of the United States. On the basis of this information, lines were drawn to show the normal coefficients of variation all over the country.

When the points first calculated and plotted seemed an inadequate basis for drawing the normal lines on the map, the Weather Bureau reports were again examined and additional records were selected to help fill in. Although shorter, and having been for that reason rejected at the time the first selection was made, these were used, until finally coefficients calculated from over 500 records had been plotted and used in defining the positions of the normal coefficients of variation. A number of Canadian records were used to aid in drawing the lines near the border.

On the whole the information was consistent. Only a very few stations were found giving coefficients so far from their neighbors as to suggest some unusual condition or error. A very few such records were excluded.

For the eastern part of the United States the records used are sufficient to show the normal conditions with a fair degree of accuracy. In the West the relative variations are greater, the records are shorter and the stations often farther apart and the results thus less reliable.

The map made up in this way is reproduced herewith. This is to be regarded as a first rough approximation. It serves to give a fairly accurate idea of the general conditions of variation in annual rainfall in the United States, but in detail it must be expected that more ample data, or even more complete use of existing data by extending the studies to cover everything available, would result in changes in the positions of the lines. Such changes

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*Of Hazen, Whipple & Fuller, Consulting Engineers, 30 East 42nd St., New York City.

[1]Minutes of Proceedings of the Institution of Civil Engineers, Vol. XXXIX, Session 1874-5, Part I, and CIX, Session 1891-92, Part III.

[2]"Rainfall, Reservoir and Water-Supply," page 12.

are most likely near mountains and in about that third of the country nearest the Pacific Coast.

The coefficient of variation is lowest on the Atlantic Coast. That means that one can count on getting more nearly the normal rainfall each year; and on the Atlantic Coast the coefficient does not range through wide limits. If one's experience were limited to this part of the country, he might almost neglect the range and assume that there was everywhere the same chance of a very dry year, such for example as is represented by a rainfall of only 60% of the normal. This is in general the conclusion reached by Binnie for the data studied by him. It does not hold, however, for the whole United States. As one goes west, the coefficient of variation in annual rainfall increases; that is to say, the chance of a very dry or a very wet year (relatively) is increased.

At New York the coefficient of variation is 0.15; at San Francisco it is 0.30. This means that on an average the relative variations in annual rainfall at San Francisco are twice as great as at New York. A year 40% short of the normal rainfall comes as often at San Francisco as one 20% short of the normal comes in New York.

By following this method of investigation it will not be hard to find quite definitely just how often, on an average, a year of any assumed relative wetness or dryness will come, and also to find what the chances are of years of any degree of dryness coming within a given period.

If the rainfall records followed exactly the "normal" law of error, a very simple procedure could be outlined that would lead directly to the result; but the law that governs the rainfall is just a little different from the "normal" law of error, and the difference is enough so that it is not best to attempt to make the calculation by its use. At present a graphical solution using probability paper (see Am. Soc. C. E., Vol. LXXVII, p. 1539) seems to be the surest way of handling the matter, and the safest way to get results applicable to a particular locality is to use the local data.

Other things being equal the variations to be expected in any period at different localities will always be in direct proportion to the coefficients of variation in the respective places.

The work of making the computation and of drawing the map was mainly done by R. H. Eurich.

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Solving the Problem of Local Engineering Associations

What seems to be an excellent solution of some of the problems noted in the editorial, "Coöperation or Amalgamation of Local Engineering Societies," in *Engineering News*, Nov. 25, 1915, is contained in the new regulations of the Associated Engineering Societies of St. Louis, which is composed of the Engineers' Club of St. Louis, the St. Louis Association of Members of the American Society of Civil Engineers, and the St. Louis branches of the American Society of Mechanical Engineers, American Institute of Electric Engineers and the American Society of Engineering Contractors. Other local organizations of 50 or more members which are branches of national engineering societies are eligible for affiliation.

The affairs of the association are conducted by a joint council consisting of the president and two junior past-presidents of the Engineers' Club, together with two other councilors chosen by each of the affiliated societies, one each year for a term of two years, at least one of whom is to be a member of the affiliated society's governing committee, and at least one a member of the Engineers' Club. The president, treasurer and secretary of the Engineers' Club act in similar capacities for the council. The council has jurisdiction over all matters of joint interest.

Joint Action on Local Affairs

The societies of the association act jointly on all local public affairs presented for consideration to any one or more of the societies. Such questions are referred to the council for its recommendation, and upon a majority vote they are put to a referendum vote of the entire membership. In national affairs the recommendations of the council, if any, are printed, and if they obtain a three-fifths vote in favor, are referred to the consideration of one or more of the national engineering bodies.

Each society is to arrange per calendar year two technical joint meetings in which all the other societies participate. One joint banquet annually and such joint meetings as may be arranged by the council are also held. Each affiliated society arranges all technical meetings other than joint meetings as "party meetings" in which only the affiliated society and the members of the Engineers' Club participate. Each individual affiliated society, however, may arrange nontechnical "individual" meetings in which it alone may participate.

The auditorium of the Engineers' Club is available for all joint and party meetings, and the members of the Engineers' Club have the privilege of participating in the discussion of any subject under discussion. In recognition of the Engineers' Club furnishing the auditorium, library, clubroom, etc., it is recognized that the St. Louis Engineers' Club is the parent society and reserves the right to hold technical and society meetings and excursions with or without the participation of one or more of the affiliated societies.

Equitable Distribution of Costs

Dues are based on the estimated cost of the meetings: $10 for rent of the auditorium; $10 for secretary service; cost of printed notices, etc. Statements of each society's accounts are rendered quarterly by the secretary of the Engineers' Club and each society, including the Engineers' Club, pays its pro rata share of the cost of meetings, based on the total number of members on its mailing list, and in the case of affiliated societies the number of members on its mailing list who are not members of the Engineers' Club. For individual meetings a rental charge of $10 for the auditorium is made.

In consideration of the Engineers' Club offer to share its clubroom, it is agreed that each affiliated society shall encourage its members to join the club and that the Engineers' Club will encourage its members to join the national engineering societies.

The papers of the various organizations may or may not be published in the monthly journal of the Engineers' Club; but there is to be published each year a Year Book of the associated societies, for which each society pays in proportion to its membership.

Any affiliated society which fails in its obligation after being notified by the council may at the end of three months be dropped from the association by a majority vote of the council. Any associated society may withdraw from the association after three months' notice.

Building the Chesapeake & Ohio Northern Railway

SYNOPSIS—Total excavation 3,600,000 yd., of which 25% is rock. Cuts and fills 90 ft. deep. Special methods of cutting. Spreaders used on rock fills. Hauls up to 5½ mi. Gumbo caused slides in cuts and proved unstable in fill; how trouble was cured. Difficult movement of heavy plant.

Heavy rockwork is met in the construction of the single-track Chesapeake & Ohio Northern Ry. between Sciotoville and Waverly, Ohio, on the first 7 mi. north of the Ohio River. Over the 28 mi. of new line four general contractors and about a dozen subcontractors are cutting and filling, and erecting culverts and a viaduct. As originally proposed the railway was to run from Columbus to Edgington, Ky., where it was to connect with the Chesapeake & Ohio Ry. The line was located between these two points; but owing to financial conditions, at the present time only the 38-mi. strip is being constructed between Limeville (2 mi. west of Edgington) and Waverly, at which point the line has track connection with the Norfolk & Western Ry., which continues to Columbus and there connects with the track of the Hocking Valley Ry. (extending to Toledo). Thus by constructing the Chesapeake & Ohio Northern, the Chesapeake & Ohio Ry. obtains a desirable outlet to the Great Lakes.

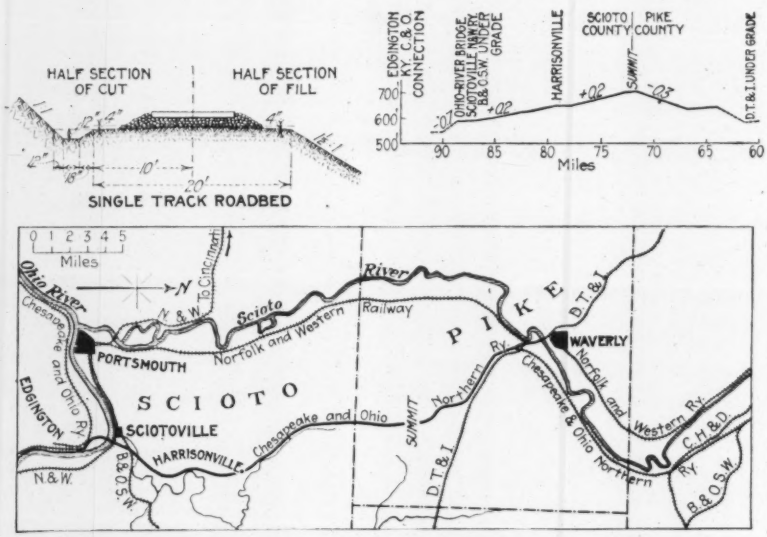


FIG. 1. MAP OF NEW LINE, SHOWING ADJACENT RAILWAYS

The maximum grade and curvature of the new line is 0.2% compensated against northbound traffic and 0.3% against southbound, with the exception of a northbound pusher grade of 0.7% on the Kentucky approach to the Ohio River bridge. Sidings 6,000 ft. in length are provided every 5 mi. They are flattened out to facilitate starting heavy trains from standstill on the siding.

Contractors—Work They Must Do

The building of the new line is supervised by the railway under three residences and is divided into four contract sections. Starting at the Kentucky end the general contractors are: Section 1 (3 mi.) W. W. Boxley, of Roanoke, Va., having 450,000 yd. of earthwork and 1,500 yd. of reinforced concrete; Section 2 (7 mi.), Rinehart & Dennis, of Charlotteville, Va., having 700,000 yd. of rockwork, 600,000 yd. of earth and 25,000 yd. of concrete; Section 3 (9 mi.), Winston & Co., having 80,000 yd. of rock, 800,000 yd. of earth and 10,000 yd. of concrete; Section 4 (9 mi.), Robert Grace Contracting Co., of Pittsburgh, Penn., having 120,000 yd. of rock, 850,000 yd. of earth and 15,000 yd. of concrete. The Dravo Contracting Co., of Pittsburgh, built the substructure of the Ohio River bridge; the McClintic-Marshall Construction Co. will erect the superstructure. A considerable part of the work is sublet.

The total excavation is estimated at 900,000 yd. of rock and 2,700,000 yd. of earth, Section 2 having 80% of the rock. Included in the construction are: The Sciotoville two-span continuous-truss bridge over the Ohio River, 1,550 ft., with viaduct approaches of 700 ft. on the Ohio side and 1,100 ft. on the Kentucky side (foundations completed having 27,000 yd. of concrete); a 1,080-ft. viaduct over the Little Scioto River; an overhead crossing of the Baltimore & Ohio Southwestern Ry.; 12 highway undercrossings; 6 overhead crossings; 3 grade crossings; numerous concrete culverts; 2 timber trestles on the temporary connection with the Norfolk & Western and a number of strips of county and township road changes. Concrete work amounts to 86,000 cu.yd., of which 32,000 yd. is reinforced; 14,500 tons of steel, including 12,200 for the Ohio River bridge is required. The first shovels started work in May, the preliminary work having been completed that same month. Work will continue through the winter, and it is not expected to be completed for at least a year.

How the Work Is Being Done

On the Kentucky side of the Ohio River a 52-ft. fill containing 450,000 yd. of borrow-pit earth forms the southern approach to the Sciotoville bridge. Record work is being accomplished at the borrow pit by a Marion 61 (2½-yd. dipper); an average of 43,000 to 44,000 yd. is being excavated per month. The biggest month for this shovel, with one 11-hr. shift working 6 days per week, was 53,000 yd. The fill is transported from the pit in 6-car or 7-car trains of 12-yd. Western air-dump cars drawn by 60-ton locomotives on a standard-gage track. A siding and a water tank are provided at the pit. The haul is 1½ mi., and two trains transport the material from the pit to the fill.

The filling trestle is 23 ft. high, having the usual two-pole bents spaced 6 ft. apart. It is capped with ties, and between bents an extra tie is spiked to the rail. By the middle of November the trestle had been filled to the top, and the track is now being jacked by track jacks as the fill rises. A western spreader is used.

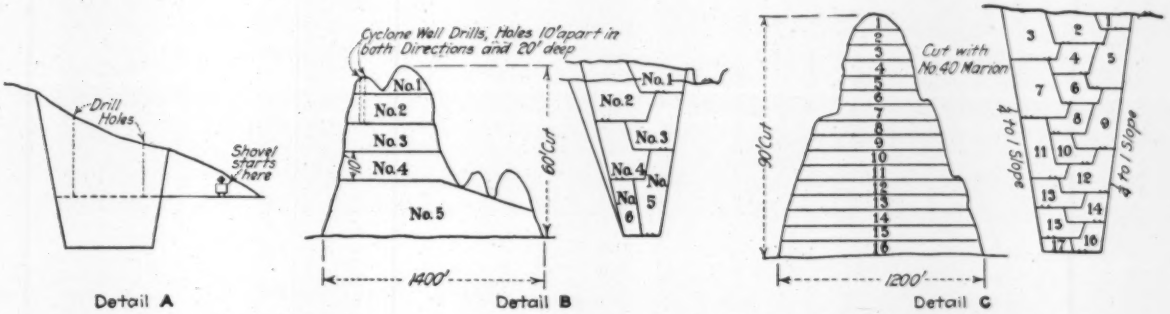


FIG. 2. METHOD OF EXCAVATING DEEP ROCK CUTS ON THE CHESAPEAKE & OHIO NORTHERN RY. WORK

Just north of the Ohio bank an 88,000-yd. semi-sidehill cut 55 ft. deep in rock is being taken out. A cross-section of the cut is shown in Fig. 2. As indicated, although really a through cut, it is being worked as a sidehill cut.

Drill holes were put down from the top by well drills. The holes were spaced about 25 ft. apart. They were first sprung with three springing charges of 15, 100 and 200 lb. of dynamite respectively; the blasting charge averaged 2,000 lb. This was tamped with clay by long wooden poles. The heaviest blast consisted of 11,000 lb. of dynamite placed in five 65-ft. holes. The rock is Waverly shale.

Clearing rock from this sidehill cut and from two more near-by (225,000 cu.yd. together) are two shovels—an 80-C Bucyrus (3½-yd. dipper) and a Marion 61. It is remarkable what large pieces can be manipulated by the shovels. The heaviness of the work is attested by the big rocks on the adjoining fill. Rocks of 14 cu.yd. have been worked around until the dipper gets a purchase on them, when they are lifted—only a point being in the shovel—and deposited in the cars.

Kilbourne & Jacobs air-dump cars of 18 yd. each transport the rock to the fills, and there a Jordon spreader, supplied by air from the locomotive, levels it. To clear the cars of large rock, it is necessary to remove one side of the car. This is a time-consuming task, as the cars are very heavy, but accidents have proved the necessity for this precaution. Two of these cars rolled down a 200-ft. bank into the Little Scioto River on one occasion when this practice was not adhered to. The wrecked cars had been insured for $1,000 each. The insurance was collected, and the cars will be rebuilt at a later date by the contractor. The material trains run on a standard-gage track. For long hauls seven or eight cars make up a train; on short hauls or steep grades three or four cars are sufficient. Langhorne, Langhorne Co. & Sneed are working these three cuts and fills.

The line crosses over the Baltimore & Ohio Southwestern Ry. a half mile south of the Little Scioto River on a 26° skew. The main track and a siding are carried over on 107-ft. deck girders 35 ft. above the Baltimore & Ohio Southwestern track. The abutments are carried on piles driven into blue clay. They were completed in December. The subcontractor on this work is C. Y. Lygon & Co., of Prestonburg, Ky.

Meanwhile a temporary trestle had been thrown over the Baltimore & Ohio Southwestern Ry., so that rock could be carried across to the fill, which extends from the crossing to the Little Scioto viaduct. This fill contains 220,000 yd., of which 50,000 is borrowed earth and the rest is rock.

The viaduct over the Little Scioto River is 1,080 ft. long. Its concrete pedestals and abutments have been completed. Upon them steel towers will be erected. The first (south) four pedestals and the south abutment bear on compact clay; the remainder are either directly on rock or are on piles driven to rock. The contractor on

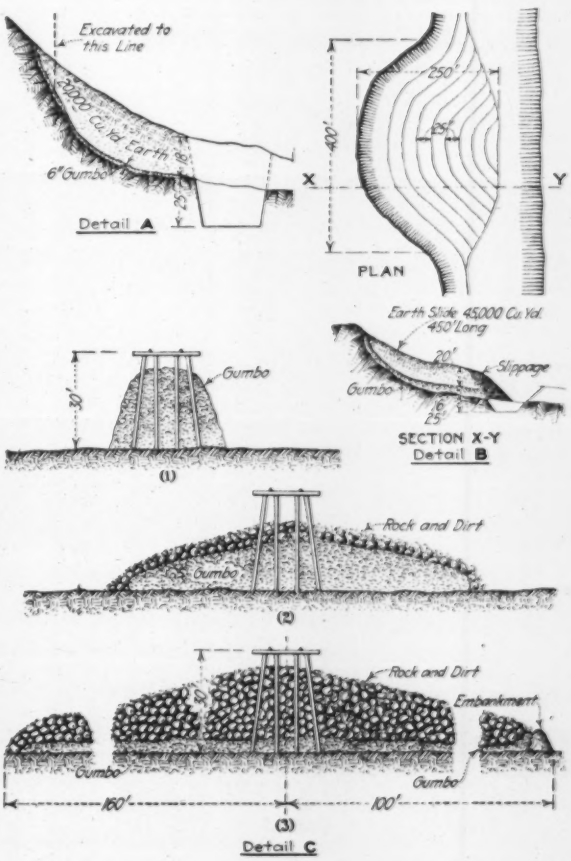


FIG. 3. EXPLANATION OF SLIDES CAUSED BY GUMBO

this work is the same as on the Baltimore & Ohio Southwestern Ry. crossing just described.

The extensive rockwork made blasting in the cuts almost universal on the southern half of the line. Drill holes are put down about 20 ft. and are spaced 10 ft. apart in both directions. Drilling, blasting and excavating proceed simultaneously. The drilling is carried on where the shovel will make its next trip or directly ahead or behind the shovel. The holes are first sprung by shooting 15 to 20 sticks (about 10 lb.) of dynamite. The final or blasting charge averages 1 lb. per cu.yd. of rock. Shooting can continue 100 to 150 ft. ahead of the shovel. Blasting is carried on as closely as possible to the permanent slope line.

The number of shovel trips through typical 60-ft. and 95-ft. rock cuts is shown in Fig. 2. As there was very little water encountered in the rock cuts, the shovel cuts were horizontal instead of being inclined for drainage.

Special Cutting Methods Employed

A "razor edge" hill, which must be cut to a depth of 90 ft., is encountered on Residency 1. The cut was started at grade and will be only 300 ft. long. The broken rock is loaded by hand on mule-drawn cars running on a narrow-gage track. The work is being done by the Perkins Construction Co. and should be finished by the first of the year.

An unusual method is being employed in the case of a cut 1,200 ft. long by 95 ft. deep at maximum. The hill has a hump on its south slope about 400 ft. long. A 6x10-ft. tunnel has been driven through this hump at grade on the slope line (on the left, looking north) and timbered. Holes are put down by churn drills and shot off in 15- to 20-ft. lifts; thus the blasting continues on down as close to the roof of the tunnel as is deemed safe. The aim is to break up the rock all the way down to the roof, but not to break through into the heading. Where the holes do not penetrate deep enough, dobey shots are necessary. In one or two instances the timbering was blown down and the heading had to be cleared of débris. After several shots were fired, cars were run into the headings, the timbering was removed (beginning at the toe of the slope of broken material from shot) and the broken rock above was raked down into the cars.

After all the rock above the heading is cleared away, the cut will be shot to width and a shovel will be put to work. The remainder of the cutting is done as illustrated by Fig. 2. The hump was blasted by a subcontractor for W. H. H. Allen, who in turn is a subcontractor for Rinehart & Dennis. From 6 to 12 men (usually 6) do the work. This method was employed because of the fast progress made by these same men on a similar job elsewhere. It is said that they there removed 6,000 yd. per month. On the present job they are averaging only 2,000 yd., owing to the heavy going.

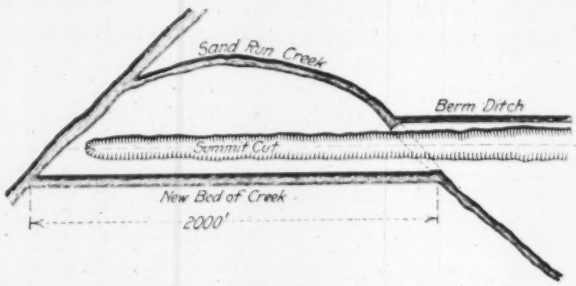


FIG. 4. CREEK DIVERSION TO ALLOW EXCAVATION OF SUMMIT CUT

A revolving steam shovel mounted on caterpillar wheels is a very novel feature, introduced by George Maxfield, a subcontractor on Residency 3. This machine, an 18-B Bucyrus with a ⅞-yd. dipper, is employed on highway-change work. The cuts were many and shallow, and in order to save on moves the shovel was mounted on caterpillars. The scheme has been a success.

A feature of this work is the long hauls in certain instances. There are two 5-mi. hauls and one 5½-mi. Up to 3 mi., 8- to 16-car trains of 4-, 4½-, 5- or 12-yd. dump cars on narrow-gage track, drawn by 20- to 70-ton dinkeys, are employed. On the long hauls stand-

FIG. 5. TRAPPING METHOD OF EXCAVATION—TUNNEL IS DRIVEN; MATERIAL ABOVE IS BLASTED DOWN TO TOP OF TUNNEL, AND TOP TAKEN OFF TO LOAD CARS

ard-gage trains are used. On some of the shorter hauls on Residencies 2 and 3, dump wagons are employed.

An especially difficult movement of equipment was accomplished by the C. W. Marshall Construction Co., of Kansas City, Mo., when it moved its plant from Lucasville on the Norfolk & Western Ry. to the work near Harrisonville, about 9 mi. distant. Three traction engines and a 5-ton White dump-body truck were employed. Almost at the start a 15% grade 700 ft. long was encountered against the movement. The shovel was stripped of its boom, dipper handle and dipper, which were placed on house-moving trucks and drawn up the hill by a traction engine. The shovel proper was placed on heavy rails and sent up under its own steam. The dinkeys were mounted on house-moving trucks and drawn by two traction engines each. The 5½-yd. dump cars were transported by the 5-ton White truck, one at a time. Three trips were made in a day of 10 hr., or a total of 55 mi. per day, 13 days being required for the total movement. The moving gang consisted of 2 foremen, 18 men and 2 teams. The truck is still employed to convey coal for the plant and carries the men and supplies to and from the job.

The Jones-Gray Co., Inc., also performed a difficult feat of plant moving when its outfit was taken from Given, on the Detroit, Toledo & Ironton R.R., to the job, about 7 mi. distant. Temporary track was built. Four locomotives, two steam shovels, one spreader, and three 12-yd. dump cars loaded with blacksmith and machine-shop supplies traveled in one train, all chained together. A shovel was placed on each end of the train facing outward, with the locomotive in the middle. Wire cables around the shovel drums helped the train uphill or eased it down. On heavy grades lines were thrown out by both shovels. Both the shovels and the locomotives traveled under steam on these grades.

The weight of this train made it necessary either to reinforce the bridges along the line or to build temporary crossings of cribwork and stringers. The train took 15 days to reach the job. In addition to this train, there were forty-nine 12-yd. dump cars which were transported in a truck hauled by a 22-hp. Buffalo Pitts traction engine.

The plant of Rinehart & Dennis, Section 2, consists of seven steam shovels, one 80-C Bucyrus, two 70-C Bucyrus, one 65-C Bucyrus, one Marion 61 and two Marions of model 41. Four of these shovels were moved under their own steam from Slocum station on the Baltimore & Ohio Southwestern Ry. from 3 to 6 mi. (with the same system used to move back in cuts), sometimes

FIG. 6. WELL DRILLS WORKING IN STRATIFIED CUT ON RESIDENCY NO. 3

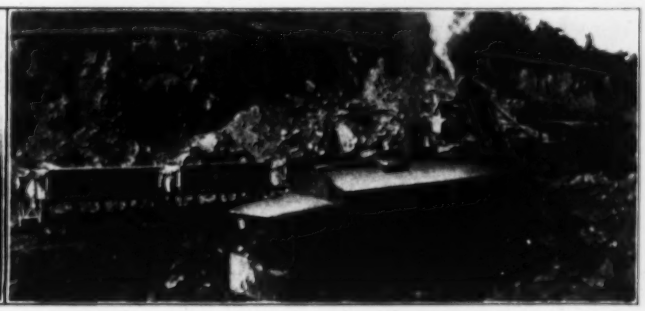


FIG. 7. START OF 45,000-YD. SLIDE IN CUT; SHOVEL CLEARING AWAY

FIG. 8. SHOVEL PARTLY BURIED FOLLOWING BLAST IN SIDEHILL CUT

FIG. 9. TAKING OUT A THROUGH CUT IN ROCK; SHOVEL IN BACKGROUND

coupling two or three of them together. One was moved from Sciotoville along the county road a distance of 6 mi, the same way. The other two were near the work on the Baltimore & Ohio Southwestern and the Norfolk & Western railroads and only required to be moved up on the cuts.

Eight 18- to 60-ton dinkeys and 50 dump cars were moved in over the same route as the shovels. This required moving them up and down grades as high as 20%, which was accomplished with the aid of cables to pull with or hold back. This plant had to be transported across the Little Scioto River. The highway bridge naturally could not be used, and it was necessary to go down the bank some 35 ft., cross in the bed of the stream under the bridge and back up the other side. One of the 60-ton locomotives, a steam shovel and the 18-yd. dump cars now working on the first two cuts north of the Ohio River bridge had to be taken in from the river edge and up a 14% grade on a very sharp curve of probably 50°. This removal was accomplished by first sending the shovel up, pulling itself by cable from drum to hitches on trees. The locomotive was pulled (under steam) by the shovel with a cable from the drum. The dump cars were pulled up by cable.

Four Spreaders Used; Two Creeks Diverted

Four spreaders are used by three different contractors. The first is a western spreader, two are Jordans, and the fourth is a home-made machine built by the Jones-Gray Co., Inc. All four have given excellent service. The two Jordan spreaders are all-steel (except floor) and will handle any rock that the shovel can load on a car. With hand spreading the rock must be broken up into one-man size, which means a lot of additional labor. The spreader works a width of 18 ft. on each side and can work as low as 18 in. below rail. Thus even for a double-track fill the construction track on top does not have to be shifted to one side, as is necessary where shovel gangs are spreading.

Summit Cut is the longest, and the spoil to be removed amounts to 700,000 yd. The deepest point is 55 ft. Along the first 1,500 ft. of the proposed cut, and crossing it, was a small creek, the drainage area of which is about ¾ sq.mi. To cut through the creek it was necessary to divert its course, as shown in Fig. 4. The same problem was met north of Summit Cut, where the drainage area was smaller. In each case the old creek bed served thereafter as a berm ditch, the regular berm ditch connecting with it at each end.

Sliding Ground in Cuts and Fills

A number of slides have been encountered in the various cuts, of which some were of serious proportions. No halfway measures of relief were taken, however, and the slides were overcome by removing all the sliding material in the zone of slippage. Fig. 3 shows the reason for the largest two slides, which occurred on Section 2. In detail A it is seen that a rock pocket was filled with earth about 18 ft. deep. Between the rock and the earth was a 6-in. layer of gumbo, on which the earth slid as soon as the shovel had gone down sufficiently. After the shovel had removed the first lift, a small slide occurred, which increased in proportions as the depth of the cut increased. Finally it became necessary to remove all slipping material, which amounted to about 20,000 yd. This was accomplished in the same general manner as shown in detail C for the second and even larger slide. In the case of the second cut there was also a pocket, but between the earth and the rock was a 16-ft. layer of gumbo, on which the earth slid with great facility. The portion of this sliding stratum that had to be removed by shovel contained about 45,000 yd. of material and was 450 ft. long by 20 ft. in maximum depth.

On the Chesapeake & Ohio Northern work, as elsewhere, gumbo was found to be a poor material for fill. All of it is now wasted. The action of gumbo in a fill is illustrated in detail C, Fig. 3. This fill is on Section 2. As shown, the gumbo held up pretty well almost to the top of the trestle, when it began to subside. Rock and dirt were then dumped upon it, and it continued to

FIG. 10. BUILDING FIRST LIFT OF 90-FT. ROCK FILL IN DAN WHITE HOLLOW

flatten. The gumbo acts as a conveyor and carries the rock down the slope and around the toe. It is hoped that the rocks, which are piled up under the toe, will work in and mix sufficiently with the gumbo to stop the slide eventually. An earth embankment has been built on one side of the toe, as illustrated; against the embankment, rocks and stones have piled up, and the lateral movement seems to have stopped. When the slipping has ceased, a second trestle will be built on top of the fill in order to bring the fill to the full height. The other method of building a high fill is, after the first lift has been made, to jack up the track until full height is attained.

Another poor material that is being wasted resembles quicksand; it contains red, white and blue clay. This material overlies the rock to a depth of about 8 ft. and

carries flowing water. Above this poor material is a 20-ft. stratum of red clay, which is excellent for filling. The wasting of this treacherous material and of the gumbo was not without its difficulties. At one point it was decided to waste into a ravine, moving over the dumping track as the fill widened, but when this was done, the track settled to such an extent that it was unsafe to allow cars upon it. The best scheme for wasting now seems to be dumping on the rock fills. The poor material fills the voids between the rocks and runs down the sides. This scheme provides plenty of space for disposing of the gumbo and from a contracting standpoint has but one disadvantage—it often necessitates long hauls, which of course represent a complete loss to the contractor.

M. J. Caples, vice-president of the Chesapeake & Ohio Northern Ry. and of the Hocking Valley Ry., is in general charge of the work. W. Michel is Chief Engineer. The resident engineers are: H. B. Watters, Sections 1 and 2; W. L. Roller, Section 3; J. W. Holman, Section 4.

Ballasted-Deck Pile Trestles; Kansas City Southern Ry.

Pile trestles built of creosoted timbers are used extensively on the Kansas City Southern Ry. They have ballasted decks to give increased life and reduce the danger of fire. The construction is shown in the accompanying drawing.

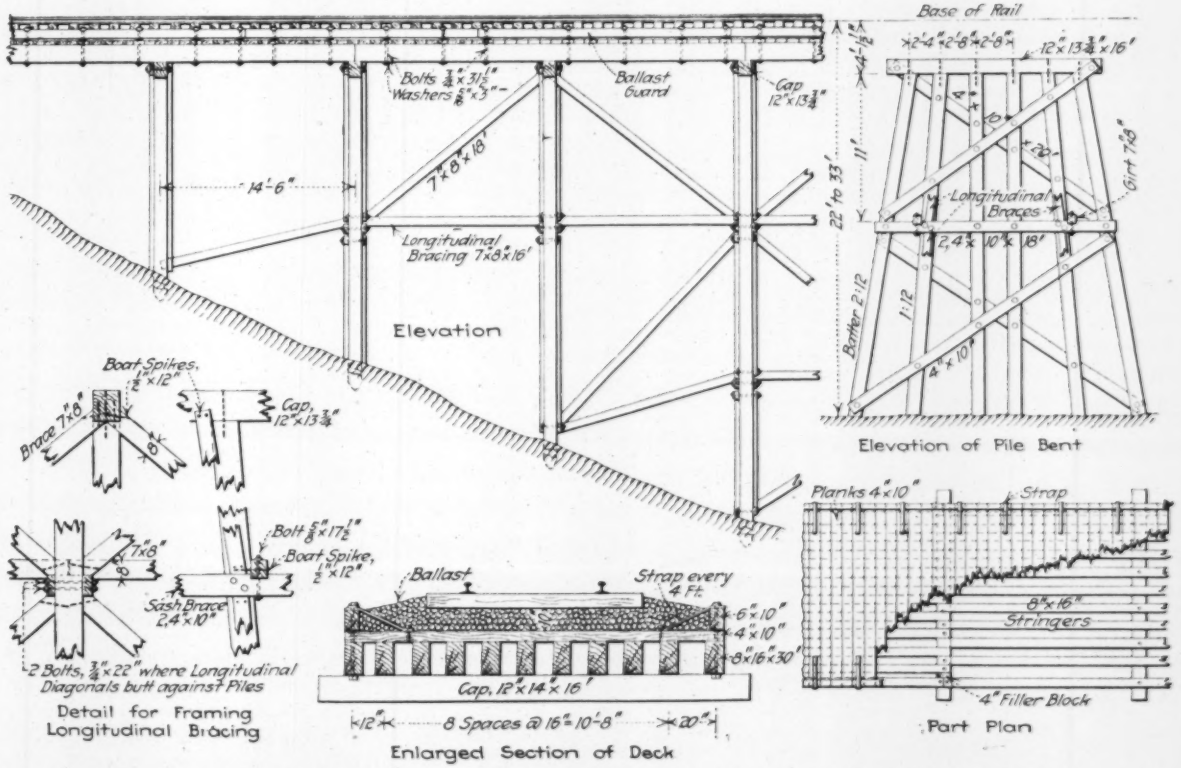
The trestle design is a good example of modern practice in a type of structure that has been a notable feature of American railways since their earliest days. Bents over 22 ft. in height have horizontal sash braces 11 ft. apart with swaybracing between them. Upon the sash braces are bolted girts or horizontal longitudinal timbers. Diagonal longitudinal braces are fitted in each panel, except in those bottom panels where they might form an obstruction to the free passage of drift.

The end bent has five piles, and back of the cap and stringers are three 6x10-in. header planks to hold the end of the roadbed. In trestles having an odd number of panels, one end panel has all its stringers 15 ft. long.

The caps and stringers are sized to 12x13¾ in. and 8x15¾ in. before creosoting. The treated timbers are handled so as to obviate cutting as far as possible, but where they have been cut in framing, the fresh surfaces are given three coats of hot creosote oil. Boltholes bored through are filled with the oil, and the bolts are coated with creosote before being placed. If the hole is not used it is closed with creosoted plugs (after the oil filling). Where spikes are removed, the holes are filled with oil and plugged in the same way.

The ends of the deck stringers are lapped on the caps, except that the outside stringers are fitted together over alternate caps. The amount of gravel required for ballast is about 0.233 cu.yd. per lin.ft. of trestle.

An earlier construction of solid deck (with creosoted timbers) was for fireproofing purposes. This had transverse floor planks laid upon the ordinary packed stringers and covered with 2 in. of chats or washed gravel as a protection against sparks and hot ashes. The rails were spiked to the floor. The new standard trestle design with ballasted floor, as here described, is more satisfactory and gives a better track. It was designed under the direction of C. E. Johnston, Chief Engineer of the Kansas City Southern Ry.



PILE TRESTLE OF CREOSOTED TIMBER, WITH BALLASTED DECK; KANSAS CITY SOUTHERN RY.

Underground-Water Dispute in California Cities

By Kenneth Q. Volk*

A controversy has developed between the cities of Riverside and San Bernardino, Calif., over their respective available water-supplies. San Bernardino is trying to enjoin Riverside from diverting water from the San Bernardino artesian basin. As the amount of available water is the measure and the limit of the development of these semiarid regions, this supply is of vital interest to these communities.

The runoff from 425 sq.mi. of high mountain drainage area discharges onto absorbent gravels in the eastern end of the San Bernardino valley, and these waters percolating seaward come in contact with what is known as the Bunker Hill dike, near Colton, where artesian conditions are produced. The artesian cap in this instance is a clay of more or less continuous nature.

The people of San Bernardino County obtain their local water-supply both for the town and for agricultural purposes by drilling wells through this clay cap and directly applying water to their lands and pumping it to their houses. In years gone by these artesian pressures at San Bernardino were sufficient to furnish domestic pressure throughout the house, but as the agricultural development has progressed these pressures have decreased. There are now 50,000 people and nearly 80,000 acres of land served from this artesian basin and the surrounding streams which feed it.

The laws with reference to the ownership of underground water have not been completely developed and fixed in California. The old rulings have been to the effect that waters percolating underground belonged to the land, and the owner of the land could extract and dispose of them at will. The later rulings of the California courts, however, have been to the effect that this can only be done as long as it is not a trespass on the owner's grounds; in other words, that a man could not buy a 100-ft. lot, start a pumping plant and not only extract water from the gravels under his own land, but also in an invidious way withdraw water from his neighbor's land. These conditions are modified and amended by what are termed the prescriptive rights. Where water has been developed for a period of five years without protest, the right becomes fixed and one of the most perfect that can be obtained to water in California.

This particular controversy is to determine permanently the respective rights of these two counties to the underground waters of this San Bernardino valley. It has involved enormous engineering and geological studies, and the plaintiff, which was the City of San Bernardino, took six weeks' time in presenting its case, from the middle of June to the first of August. The case then was postponed with the hope that a compromise agreement could be arrived at, and efforts have been made by representatives of the different parties at interest to accomplish this, but without avail. It now appears that the case will again go to trial in the early part of April.

Extensive studies have been made of the character and extent of the artesian gravels, the rise and fall of the water plane, the limitations of the artesian areas and available water-supply, as well as the legal rights.

*Engineering Offices of J. B. Lippincott, Central Building, Los Angeles, Calif.

Reducing Hillside Street Grade from 29 to 16 Per Cent.

On account of the fine views obtainable from their slopes some of San Francisco's hills are popular residence sites. Unfortunately, however, little thought was given to topography by the surveyor who in 1849 first laid out the rectangular street system, which results in grades of 55% on some of these side-hill streets. To eliminate at a minimum cost as many as possible of these excessive grades has been the recent policy of the city. No standard plan has been adopted, but a separate study is made of each case.

An interesting regrade problem recently undertaken at the request of the property owners affected is the redesign of Larkin St. between Chestnut and Francisco St., which reduces the grade from 29% to a maximum of 16%. The cost of the proposed improvement will be paid for by the owners having frontage on the streets to be regraded. On Larkin St. $16.31 per front foot will be assessed against the property on each side of the street, and on Francisco St. the rate will be $20.29 per front foot. Present property values on both these streets are now in the neighborhood of $115 per front foot. The regrade will probably cause an advance in value to $200.

Regrades on Larkin Street

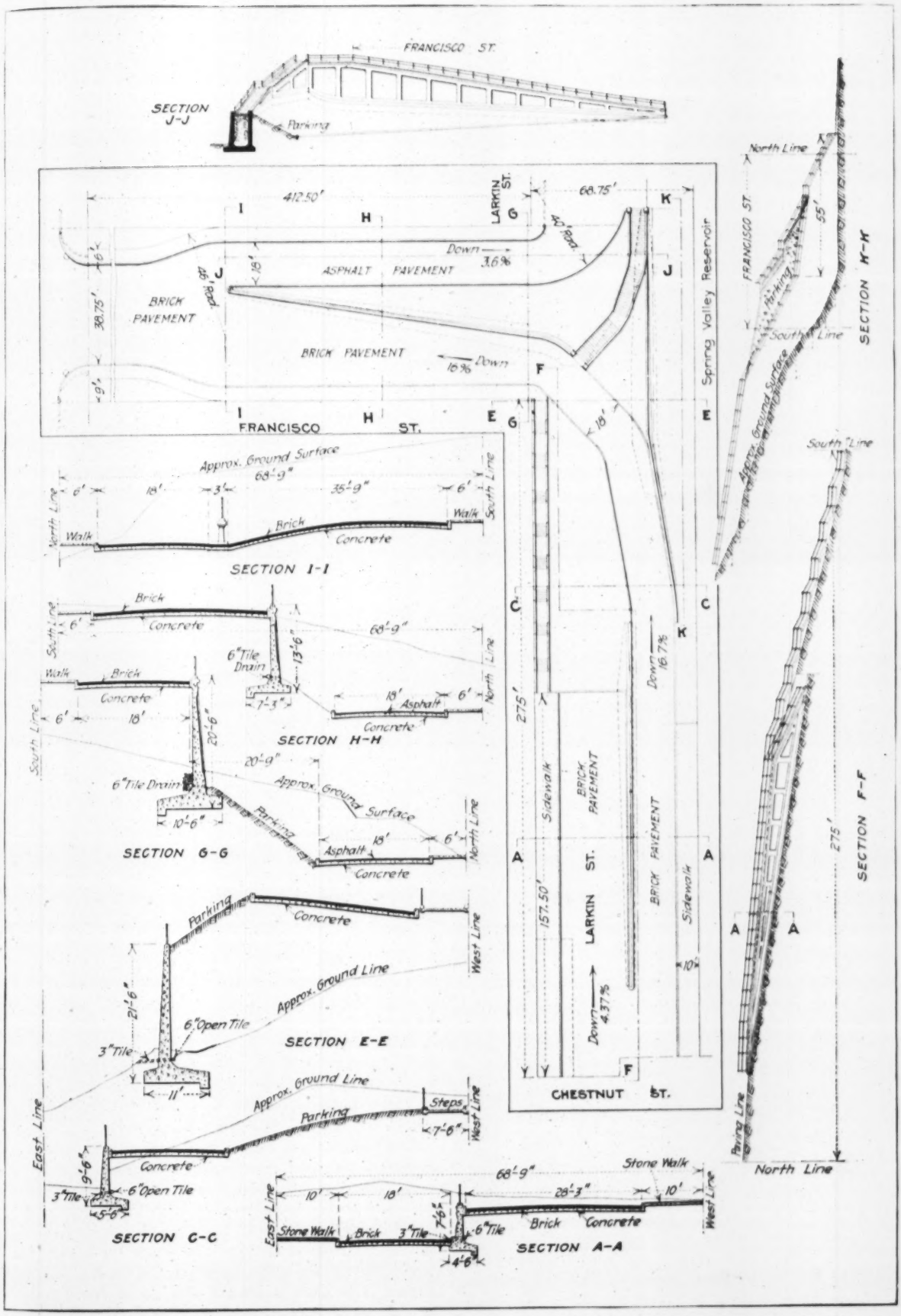
For a width of 28 ft. the east side of Larkin St. (see accompanying plan) between Chestnut and Francisco St. will be cut down to descend on a uniform grade of 16.07%. On this incline will be an 18-ft. vitrified-brick roadway and a 10-ft concrete sidewalk. This strip will be separated from the west side of the street by a reinforced-concrete retaining wall, and near the bottom of the hill will turn into the south side of Francisco St.

The west side of Larkin St., which will have a higher elevation than the east side, will descend on grades varying from 4.37 to 10.92% for a distance of 157.5 ft., where a vitrified-brick pavement 28.25 ft. wide will terminate in a parked slope. At its termination the west roadway is 12 ft. higher than the east roadway directly opposite. The west sidewalk will continue down the west side in a series of steps to the level of the east roadway at Francisco St.

Turning westerly the descending east roadway will continue on a 16% grade along the south side of Francisco St. for a distance of 127.5 ft., separated from the north side of the street by a reinforced-concrete retaining wall. Around the westerly end of this wall the roadway turns through 180° and descends easterly along the north side of Francisco St. on a 3.6% grade back to Larkin St., into which it turns northerly.

The sewerage arrangement consists of two 8-in. pipe sewers, dead-ended at Chestnut St., descending under each of the pavements on Larkin St. to a point opposite the beginning of the stairway. Here the westerly 8-in. sewer is connected to the easterly 8-in. sewer by a 12-in. pipe. The combined sewers are a 12-in. pipe turning to the west on Francisco St. The underdrains of each retaining wall are connected to these sewers through catchbasins. In profile the sewers are parallel to the new street surfaces.

The contract for this construction work was awarded to F. Rolandi at an estimated cost of approximately $30,000. The construction will be done under the direction of M. M. O'Shaughnessy, City Engineer.



PLAN, ELEVATIONS AND CROSS-SECTIONS OF THE REDESIGN OF PART OF LARKIN ST., SAN FRANCISCO

Eight Years of Imhoff Tank Design and Operation--I*

By Karl Imhoff†

SYNOPSIS—The engineer who developed the form of two-story sewage-settling tank or modified septic tank bearing his name reviews eight years' German and American experience in design and operation of Imhoff tanks. The translator, who worked with Dr. Imhoff in Germany and was subsequently engaged on the design of Imhoff tanks for Baltimore, Md., has added a number of supplementary notes.

Form and Dimensions of Tank

The form of sewage-settling and sludge-digesting device known in Germany as Emscherbrunnen and in America as Imhoff tanks was first introduced eight years ago. Today there are many municipalities making use of it. In Germany there are 170 plants designed to treat the sewage of 3,000,000 people. Of these plants 130 are in operation and 40 are under construction. In the United States 70 plants, to care for 600,000 people, have been designed, and over half of these are in operation.

The following four questions are pertinent: (1) What form of tank has given the best results? (2) What should be the dimensions of the plant? (3) How should the plant be operated? (4) What performance may be expected?

What Form of Tank Has Given the Best Results?

In order to discuss the best form of tank it will be necessary to describe the most important forms that have been used.

Fig. 1 shows the original design of Imhoff tank: A horizontal-flow sedimentation chamber situated above one or more sludge-digestion chambers. The invert of the sedimentation chamber consists of thin partition walls which serve to direct the sewage flow but which are not required to withstand water pressure. These partition walls are inclined at such an angle that the sludge which settles out from the sewage may not form harmful deposits upon the walls but must glide off and pass into the sludge chamber through slot *a*, situated at the lowest point of the sedimentation section. This slot is overlapped so that no gas bubbles or sludge particles rising in a vertical direction in the sludge chamber may escape into the settling chamber.

The inflow of sewage into the settling chamber takes place through a gate. The distribution of the sewage is effected by a shallow baffle having a penetration of 12 in. A like baffle is situated at the effluent end of the tank. In order to maintain an approximately constant water level in the tank the outflow of sewage should take place over as long a weir as possible, which should therefore extend across the whole width of the sedimentation chamber. If there is more than one sludge chamber beneath the settling chamber the direction of the sewage flow through the tank must be reversed about once every three weeks. The bypass channel *b* is provided for this purpose.

The sludge in the sludge chamber decomposes biologically, so that after a time it loses its objectionable character. It can then be discharged through the sludge pipe *c* and conveyed either by gravity or by means of pumps to the sludge-drying bed *d*. The liquid sludge is black and has an odor somewhat resembling that of tar or rubber. A layer of sludge 12 in. deep will reduce in drying to 8 in. in about one week and has then many of the characteristics of garden earth or humus.

What is most important is that the processes of decomposition in the tank be limited to the sludge. Only then will these processes take place without obnoxious odors. It is necessary therefore to make sure that no flow of sewage takes place through the sludge-digestion chamber. As soon as appreciable amounts of sewage are permitted to pass through it such sewage decomposes and develops putrid gases similar to those developed in the well-known septic tank.

The removal of settleable solids takes place in the settling chamber entirely independently of the processes of decomposition taking place in the sludge chamber. The extent to which settleable solids are removed depends upon the time which the sewage consumes in flowing through the tank. If this time, known as the detention period, be not made too long the sewage will be practically as fresh on leaving the tank as it was when it entered.

Adding an Element of Economy

This form of Imhoff tank is the simplest as well as the oldest, and after the lapse of eight years it must still be designated as the best. Only in one particular is it possible to add an element of economy. Operation experience has shown that the surface of the sludge should not be allowed to rise above the lowest point *a*, Fig. 2, of the inclined diaphragm. This fact limits the capacity of the sludge chamber. It is possible to increase this capacity considerably if instead of designing the tank in accordance with form *B* it be designed in accordance with form *A*. It is necessary in any case to support the inclined walls by means of a longitudinal beam; this beam may just as easily be given the section shown in design *A* and be thereby made to act as a protecting gas deflector for the two slots which have now been formed from the original single slot. This form of construction was first used in 1907.

Many variations of the original form of tank have been attempted, but they cannot all be described here. For example, in the case of very wide tanks the sedimentation chamber has been designed with three slots (Goerlitz) in order to prevent too deep a penetration into the sludge chamber. In America, where the sewage contains relatively less settleable solids, the settling chamber becomes large when compared with the sludge chamber; the natural tendency therefore is to design it as wide as possible and to provide it with more slots (Fig. 3).

*Translated by Leslie C. Frank, Sanitary Engineer, United States Public Health Service, Washington, D. C.

†Division Engineer, Emscher Federation, Essen, Germany.

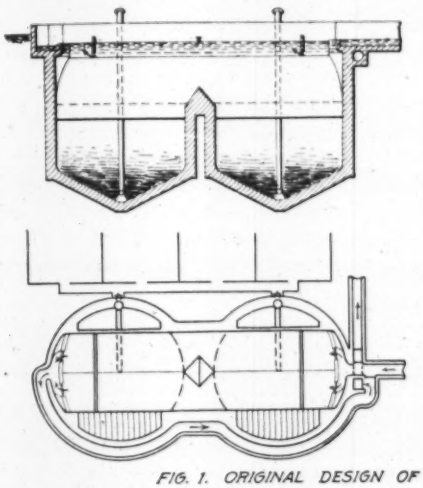


FIG. 1. ORIGINAL DESIGN OF IMHOFF TANK

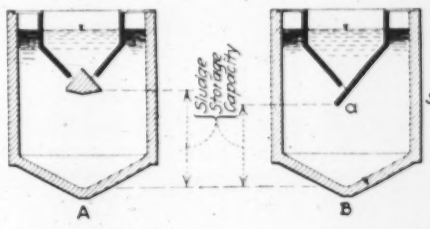


FIG. 2. EFFECT OF DESIGN ON SLUDGE-STORAGE CAPACITY (A & B)

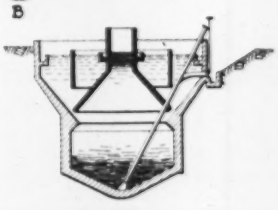


FIG. 5. DOWNWARD AND UPWARD FLOW TANK

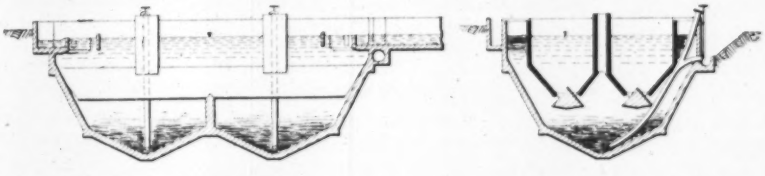


FIG. 3. FOR WEAK SEWAGE: LARGE SETTLING CHAMBER AND SMALL SLUDGE CHAMBER

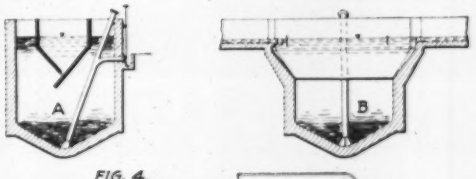


FIG. 4. HORIZONTAL EXTENSION OF SETTLING CHAMBER (A, B & C)

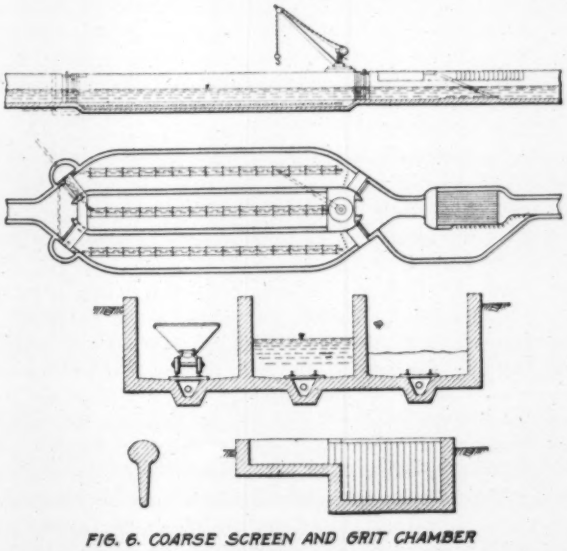


FIG. 6. COARSE SCREEN AND GRIT CHAMBER

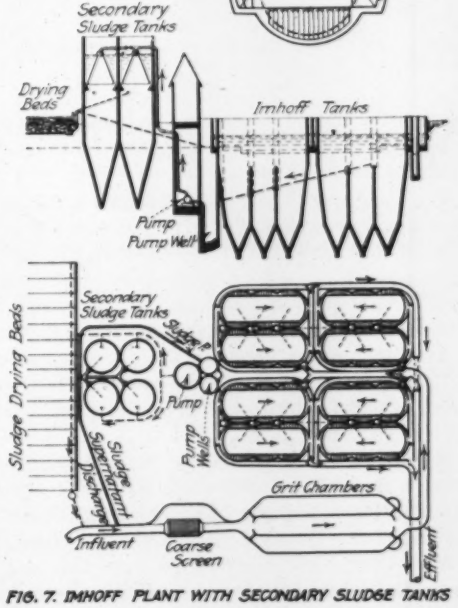


FIG. 7. IMHOFF PLANT WITH SECONDARY SLUDGE TANKS

FIGS. 1 TO 7. DEVELOPMENT OF DESIGN OF IMHOFF TANK FOR SEWAGE CLARIFICATION AND SLUDGE DIGESTION

For the same reason it becomes desirable in many cases to extend the sedimentation chamber horizontally beyond the sludge chamber (Fig. 4). Not more than two or three sludge chambers should be connected with one settling chamber, but here hard and fast rules cannot be given.

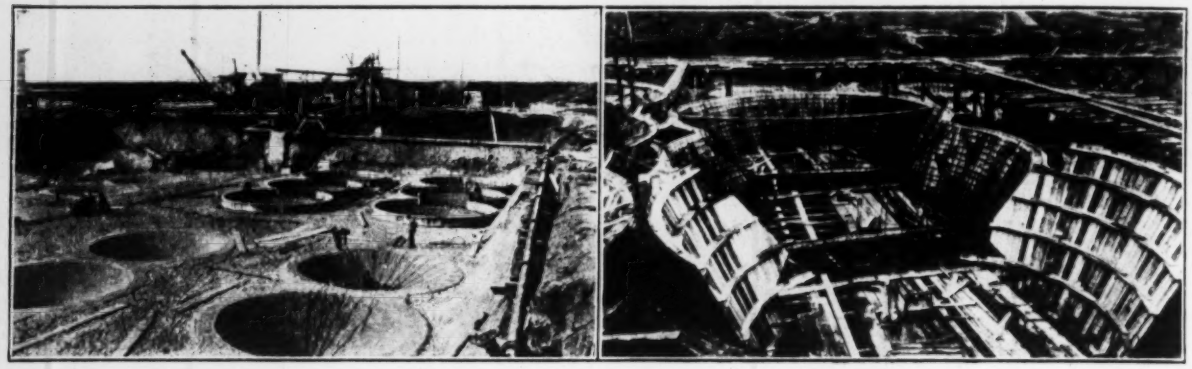
Round Chambers Generally Cheaper

In all of the accompanying illustrations only round sludge chambers are shown. It is not intended thereby to designate round sludge chambers as the best for all cases. In America many rectangular plants have been built. The American engineers justify this by the statement that contractors find difficulty in constructing round forms for reinforced concrete and therefore exact an accordingly higher price. I believe, nevertheless, that round tanks are cheaper, because the walls may be designed upon the basis of hoop tension only, since they are not subjected to bending moment. Consequently the wall sections are relatively much thinner and the amount of steel required relatively much less. I have never built other than round sludge chambers, since a cylinder would seem to be the natural shape for any water container. The round shape also makes it possible to construct the tanks in either of two ways: To sink them as caissons, or to use open-pit construction. In the first plants constructed in Germany the tanks were sunk as caissons, and this method of construction is to be recommended wherever the ground is soft. The open-pit form of construction is often possible, of course. (See the views, Figs. 8 to 13, for examples of the two forms of construction.)

Regardless of how the tanks are designed or constructed they must at least be sufficiently strong to withstand the inner water pressure without reliance on any exterior support. It is very seldom indeed that it is possible to depend for support upon the ground material adjacent to the tank walls.

All examples of Imhoff tanks thus far shown have the sewage flow taking place through them in a horizontal direction Experience has shown that this is the best method. Plants in which the direction of sewage flow is downward and upward have not given such good results. Their description in this paper will therefore be made brief. Such tanks as have been built for downward and upward sewage flow have consisted of individual round units. In them the sewage is led to the center, flows out radially and downward under a deeply penetrating baffle and then upward to the circumferential effluent channel (Fig. 5).

The advantage of this design is that it adapts itself admirably to the circular form of construction. Experience in many plants of the Emscher district has shown, however, that the removal of settleable solids is less efficient in this form of tank than in the horizontal-flow design. The suspended solids have little opportunity of being removed from the sewage during its downward course, and the upward direction assumed by the sewage, once the baffle has been passed, tends to carry with it some of the lighter settleable solids and to permit them to pass off with the effluent. The downward-and-upward design of plant is totally unadapted for sewage coming from "combined systems" if the velocity of flow through the plant during times of storm is expected to be increased. The disadvantages of the downward-and-upward system in the case of the separate system of sewerage are relatively less. I no longer recommend this type of tank under any circumstances. If it is desired to convert such tanks for horizontal flow, it is merely necessary to remove the deep baffle and to lead the sewage through the tanks in a radial-horizontal direction, substituting for the deep baffle a shallow scum ring near the outer circumference. Tanks of this latter shape have been designed and constructed for the city of Baltimore. It is possible to modify these tanks still further by leading the sewage through them from one side to the other rather than radially. Plants of this kind have been constructed in Germany.

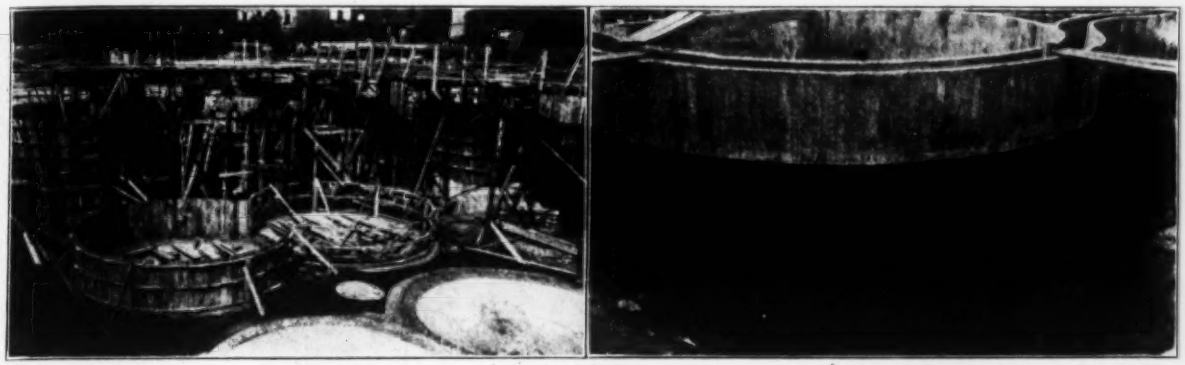


FIGS. 8 AND 9. OPEN-PIT CONSTRUCTION AND FORMING AT MEIDERICH-HAMBORN, GERMANY
Left—Sludge sumps. Right—Flare forms

Deep tanks are to be preferred to shallow tanks because in them the decomposition of the sludge is improved. This is so because in these deeper tanks the temperature is maintained more uniformly and because the stirring action of the rising gas bubbles is more intense. Because of the relatively better sludge decomposition the sludge chambers of the deeper tanks need not be designed so large. In addition the sludge-drying beds may be designed smaller because the sludge from the deeper tanks contains more gas and dries more easily.

[To the advantages of depth of tank may be added the possible advantages of depth of sludge layer. Sludge flakes are probably compressible and may be likened to very small sponges. If a piece of wire screen were made to cover a mass of water-soaked sponges in a container and the screen then loaded with weights the

sponges would compress and a water layer appear above the screen. The moisture content of the sponges would naturally thereby be reduced. In like manner any horizontal plane in a mass of sludge may be considered as being occupied by a screen and the mass of heavier-than-water sludge above this plane as equivalent to the weights. It seems probable then that if sludge particles are both compressible and heavier than water the depth of sludge layer will have an important influence upon the average moisture content of the sludge, or the deeper the layer the lower the average moisture content. This may afford another reason why with deep tanks the sludge chamber need not be designed so large as with shallow tanks. The same factor will also influence the required area of sludge-drying beds.—Translator.]



FIGS. 10 AND 11. IMHOFF TANKS UNDER CONSTRUCTION BY OPEN-PIT METHOD AT STEELE, GERMANY

Left—Sludge-chamber sump completed in front; form work for sludge chamber in middle and in rear. Right—Forms removed from top of tank

In discussing Imhoff sewage-treatment plants it will be pertinent to consider those parts auxiliary to the tanks themselves. The simplest designs have here also proven to be the best.

Coarse Screens and Grit Chambers

Coarse screens are absolutely necessary. The function of these screens should not be confused with that of so-called fine screens. The function of the coarse screens is not to remove as much of the suspended matter as possible, which is true of fine screens, but instead to remove only such coarse matters as would tend to clog the Imhoff tank sludge pipes. Above all it is highly desirable that fecal matter be not retained on these screens. Coarse screens therefore should have a clearance of about 2 in. They should be placed in an inclined position and the sewer cross-section at this point should be increased somewhat in width, but not much. The coarse screens should be cleaned by hand from time to time. Only in the case of very small plants which are close to the source of the sewage and at which the fecal matters arrive in an undivided state will it occasionally prove advisable to install a screen with a smaller clearance than above, on which these undivided fecal matters may be retained and broken up so that they will not float for too long a time upon the surface of the tank.

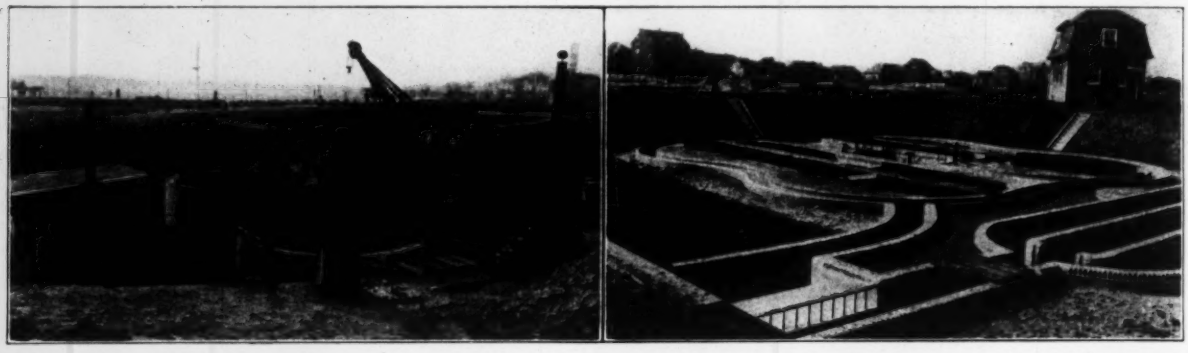
Grit chambers will be required in large plants that receive sewage from a combined system. In the case of grit chambers, as in the case of coarse screens, it is undesirable to retain appreciable amounts of organic matter. Its presence would induce difficulties in the subsequent handling of the grit. It should in fact be the aim to retain as nearly as possible only pure sand or grit. To this end a velocity of about 8 to 12 in. per sec. should be maintained in all parts of the chamber. The grit chamber ought not to be deep. It forms, so to speak, simply a modified part of the sewer, in which the velocity of flow has been reduced to below the grit-transporting point. The length of the grit chamber depends on the amount of grit which it is expected will be received during the interval between cleanings, say three days. The grit chamber may most advantageously be constructed in three units. To clean one of these units it is necessary to take it out of operation and drain the retained grit by means of the underdrainage system provided in the design (Fig. 6). The grit may then be removed by hand or with the aid of a crane.

The system of sludge pipes provided for the sludge chambers must include a pipe and a gate for each chamber, so that it will be possible to discharge from any chamber independently. The pipe should preferably discharge directly into an open channel, so that the attendant may be able to observe the sludge as it comes from the tank. The hydraulic gradient which should be provided for closed sludge pipes is about 12% and for open channels 2½%. It is advisable to include in the bottom of the sludge chamber a water-agitation system. This system should not be connected permanently with the water mains as it will be used but seldom if at all. The system is provided merely so that it may be possible, if necessary, to agitate the sludge in order to promote its decomposition. If the operation of the tank shows that such agitation is necessary the water-agitation system may be connected with the main by means of a hose.

The sludge beds should consist of a comparatively large number of independent units, so that in drying the sludge some of these units may be receiving fresh sludge while others are drying and while still others are having the dried sludge removed. The sludge-drying beds are provided with an underdrainage system that is covered with a porous material such as slag or gravel. On this gravel must be placed 1½ or 2 in. of sand or some other equally fine material. This layer of fine material must be renewed from time to time because each removal of dried sludge includes a certain amount of the sand layer. Such renewal of fine material is provided for in the plants of the Emschergenossenschaft by making use of the grit recovered from the grit chamber. Each drying bed is provided with a narrow-gage track

placed on a level with the upper surface of the bed. These tracks permit the use of sludge cars and thereby greatly facilitate the removal of the sludge.

In the first Imhoff sewage plants the pumping of sludge was avoided as much as possible because the drying of the sludge is promoted by its gas content, and pumping tends to remove this gas content. This difficulty has since been obviated. In the new plants of the Emscher district the pumps are augmented by a secondary sludge tank. This tank is of such size as to provide for the sludge a detention period of about one week—which will permit the necessary gases to form. From the secondary tank the sludge flows by gravity to the sludge bed in the same condition, so far as its drying characteristics are concerned, as if it had never been pumped (Fig. 7). In many of the more recent plants the secondary sludge tank has been made larger than above indicated so as to permit the sludge chambers under the settling chambers to be made smaller. This is simply equivalent to providing part of the required sludge-digestion capacity under the settling chambers and another part above the sludge-drying beds.



FIGS. 12 AND 13. PROGRESS VIEW AND COMPLETED IMHOFF PLANT AT WANNE-NORD, GERMANY

Left—Caisson type of construction. Right—Completed plant, showing screen chamber and grit chamber in front and tanks at rear

What Should Be the Dimensions of the Plant?

The settling chamber of an Imhoff tank should be designed like any plain sedimentation chamber. It is necessary to determine first of all what should be the detention period. This depends upon the character of the sewage and upon the degree of sedimentation required. Within limits velocity of flow is not significant. In the settling chamber, if it be properly designed, the process takes place in precisely the same manner as in any quiescent container. It is necessary only to imagine the quiescent container filled with sewage and passing through the chamber at the same velocity as the other sewage in the chamber. It is evident that the velocity of the sewage body as a whole in a horizontal direction, so long as it is not a swirling velocity, has no influence upon the process of sedimentation.

For normal municipal sewage the practice in Germany is to allow a sedimentation period of 1½ to 2 hr. If the sewage contain trade wastes of a nature to promote the process of settling, a period of 45 to 60 min. will suffice. To increase the sedimentation period beyond 2 to 3 hr. I believe to be uneconomical.

[There is very little practical difference between a nominal detention period of 1.5 hr. as used in Germany and a nominal detention period of 2 hr. as used in the United States. The German design assumes that the daily flow takes place in 14 hr., and the American design that it takes place in 18 hr. On this basis the German settling chamber would be equal in size to $1.5 \div 14$, and the American settling chamber to $2 \div 18$ times the daily sewage flow. Per volume of flow the American settling chamber is therefore only 3.5% larger than the German settling chamber. Inasmuch as the 14- and 18-hr. assumptions in Germany and the United States are both arbitrary it would seem simpler to use mean detention period instead of nominal detention period and to say that the mean detention period based on the average daily sewage flow is in most cases in both the United States and Germany about 2.6 hr.—Translator.]

If it be assumed now that the detention period has been determined, the settling chamber is then made of such a size that this detention period will be realized during times of normal dry-weather flow. By normal dry-weather flow is not meant here the maximum flow of the day, but something less than the peak load. In general it is good practice to regard as a normal hourly flow for American conditions one-eighteenth of the total daily flow. In Germany the corresponding figure is one-fourteenth. If, for example, the population of a given town is 10,000 and the sewage flow 50 gal. per head per day, the total daily flow is 500,000 gal. On the above assumption the normal hourly flow is accordingly one-eighteenth of 500,000, or 27,700 gal. If it is desired to give a settling period of 2 hr. the sedimentation chamber must then have a capacity of $2 \times 27{,}700 = 55{,}400$ gal. During times of storm flow the detention period is of course much less. By detention period here is always meant the computed detention period. The actual detention for part of the sewage is naturally less because of short-circuiting; it can be determined by means of dyes placed in the influent.

Dimensions or Capacity of Sludge Chambers

The dimensioning of the sludge chamber is less simple. For this purpose it is theoretically necessary to know how much sludge is to be expected, how far this sludge will be reduced in volume by decomposition, how long it should remain in the tank and how much space should be provided as reserve space for the accumulation of sludge during such time as it is impossible to discharge any. Practically, however, it avails little to attempt to estimate each of these individual factors in order to obtain the final result. The final result might as well

be estimated at once. Sufficient practical experience is now available to estimate the number of cubic feet of sludge capacity that will be necessary per person connected with the plant. The values given in Table 1 are to be recommended.

TABLE 1. RECOMMENDED SLUDGE CAPACITY FOR GERMANY AND THE UNITED STATES
(Cu.Ft. per Head of Population)

	Germany		America—Northern States with Long Winters	
	Separate System	Combined System*	Separate System	Combined System*
Small plants, less than 5,000 population	1.6	2.4	2.4	3.6
Normal city plants whose sewage contains some trade wastes	0.8	1.2	1.2	1.8
City plants whose sewage contains an abnormal amount of sludge-forming trade wastes	1.2	1.8	1.8	2.7

*Presupposes the use of a grit chamber.

With this table it will be possible to estimate the correct value for any particular case. Naturally a considerable latitude is assumed. Deep sludge tanks may require less than the above figures, and shallow sludge tanks more. Plants in northern climates where long winters obtain must be designed larger, and those in southern climates smaller. There are plants whose sludge chambers are designed only half as large as the figures given in the table, and which, with intelligent operation, nevertheless yield very good results. In general it is better to be generous rather than sparing in designing sludge chambers. This should particularly be observed for American conditions because there the greater portion of the cost lies in the sedimentation chamber, which is relatively much larger than the sludge chamber because of the much larger volume of sewage flow per capita.

The volume of wet decomposed sludge that is to be expected for the separate system of sewerage is 0.1 liter per head per day, and for the combined system 0.15 liter per head per day. An additional 0.15 liter or more per day will have to be allowed for cities whose sewage contains much sludge-forming trade wastes. With the assumption that the sludge-drying bed is filled each time to a depth of 12 in. with wet sludge, and that the number of times any one sludge bed is filled during the year is 15, the values shown in Table 2 will hold in determining the size of required sludge-bed area.

TABLE 2. SLUDGE VOLUMES AND SLUDGE-DRYING BED AREAS

	Ordinary City Sewage Containing Some Trade Wastes		City Sewage Containing an Abnormally Large Quantity of Sludge-Forming Trade Wastes	
	Separate System	Combined System	Separate System	Combined System
Volume of wet sludge in liters per head per day	0.1	0.15	0.25	0.3
Area of sludge-drying bed in sq.ft. per inhabitant	0.2	0.3	0.5	0.6

As an average value for the sludge-bed area ⅓ sq.ft. per inhabitant may be kept in mind. Naturally the actual area of sludge-drying beds required will depend largely upon the manner of operation and upon the individual conditions obtaining. Small plants in which it is not possible to discharge sludge so often will require a larger sludge-drying area. Large plants where the operation can be relied upon as being extremely careful and intelligent may require less than the above figures. The sludge beds in the Emscher district of Germany are smaller than the values given.

Model City-Planning Board

A city-planning board to pass upon all proposed actions affecting the plan of the city is a feature of the model commission-manager city charter now being given its finishing touches by the committee on municipal program of the National Municipal League. The city-planning article of the charter, as adopted by the committee on Dec. 27, 1915, is in part as follows:

There shall be a city-planning board of five members consisting of the director of public works and utilities and two citizen members chosen because of their knowledge of city planning. (In larger cities having a director of utilities a board of five members consisting of the directors of public works and utilities and two citizen members is recommended.) It shall be the duty of the board to keep itself informed of the progress of city planning in this and other countries, to make studies and recommendations for the improvement of the plan of the city with a view to the present and future movement of traffic, the convenience, amenity, health, recreation, general welfare and other needs of the city dependent on the city plan; to consider and report upon the design and their relations to the city plan of all new public ways, lands, buildings and bridges and all other public places or structures, as well as all additions to or alterations in those already existing and the layout or plotting of new subdivisions of the city or of territory adjacent to or near the city.

All acts of the council or of any other branch of the city government affecting the city plan shall be submitted by the council to the board for recommendations.

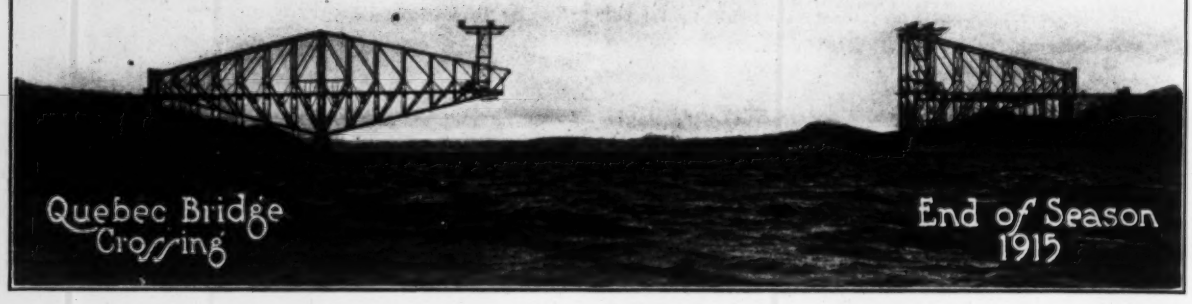
Any matter referred by the council to the board shall be acted upon by the board within 30 days of the date of reference, unless a longer or shorter period is specified. No action by the council involving any points hereinbefore set forth shall be legal and binding until it has been referred to the board and until the recommendations of the board thereon have been accepted or rejected by the council.

The board shall submit to the council an annual report summarizing the activities of the board for the fiscal year, the recommendations made by it to the council during the year and the action of the council during the year on any and all recommendations made by the board in that or former years. The annual report of the board shall also contain a program for improvements to the city plan year by year during at least the coming three years, with estimates of the cost thereof and recommendations as to how the cost shall be met.

The board shall appoint as secretary a person of skill and experience in city planning and may employ consulting city-planning experts as need may arise. The city engineer shall serve as chief engineer of the board, and it shall be his particular duty to make recommendations designed to bring all the engineering works of the city into harmony as parts of one comprehensive plan. The executive health officer of the city shall advise the board from time to time of any municipal improvements within the scope of the board which in his opinion would improve the healthfulness of the city. The board shall have power to call upon any branch of the city government at any time for information and advice which in the opinion of the board will insure the efficiency of its work.

Quebec Bridge Erection: Progress In 1915

By H. P. Borden*



Very satisfactory progress was made during the past season on the erection of the Quebec Bridge. When the work of erection ceased, about the middle of November, the north anchor and cantilever arms had been completely erected, and the south anchor arm out to and including the main post, with the exception of a certain amount of riveting in the latter, which will be finished next season.

Work on the North Shore

Work on the north side of the river proceeded during the summer very closely in accordance with the program mapped out at the beginning of the season. A start on the erection of the steel was made about the middle of April, and by the first of June the anchor arm had been completed and the main post erected and almost entirely riveted. From this point on, the erection of the cantilever arm proceeded without any hitch until completion.

Table 1 shows the progress of the work by dates and the approximate amount of steel erected. The maximum material erected by one traveler in one day was 670 tons. There were several days in which the tonnage exceeded 600. In the erection of the main post an average of over 350 tons a day was maintained. July was the biggest month, 4,200 tons being erected. This refers to one traveler and in each case on the south shore.

The ease with which the enormous members were put in place and fitted was almost phenomenal, practically no delay whatever being occasioned at any of the joints. Owing to the facts that very great care was taken in the shop during the course of manufacture and that all field splices were completely assembled and reamed out in place, no shop errors had to be corrected in the field, with the result that no time was lost in this manner.

TABLE 1. PROGRESS OF ERECTION OF CANTILEVER ARM, QUEBEC BRIDGE

Traveler at Panel Point	Date	Days Required, Each Move	Days Required, Full Panel	Approximate Tons of Steel Erected
Main post	14–16 June 5	.....	..	....
15	June 17	12 Subpanel	..	950
14	July 13	26	38	1,993
13	July 19	6 Subpanel	..	828
12	Aug. 11	23	29	1,732
11	Aug. 16	5 Subpanel	..	452
10	Sept. 2	17	22	1,450
9	Sept. 8	6 Subpanel	..	447
8	Sept. 18	10	16	968
7	Sept. 20	2 Subpanel	..	493
6	Sept. 30	10	12	747
5	Oct. 4	4 Subpanel	..	288
4	Oct. 9	5	9	538
Last move 2	Oct. 25	16	11	610
0	.....	.. ..	..	636

Accurate measurements were taken as each panel was erected, to determine whether the cantilever arm was in its theoretical position. Center marks were placed on each floorbeam before assembling, and it has been found that these marks were in perfect alignment from the end of the cantilever arm to the anchor pier.

Lifting of the North Anchor Arm

The total dead weight of the cantilever and anchor arms is 24,000,000 lb. and 26,000,000 lb. respectively, exclusive in both cases of the main post over the pier. The erection traveler under working conditions weighs approximately 1,000 tons. The first effect of the cantilevering out was noticed from a lifting movement in the anchor-arm truss when the traveler was at panel point 11, just after the bottom-chord section CL10-11 had been put in place. Subsequent movements of this truss are shown in Table 2, which gives the vertical movement of the bottom chords which were supported on steel falsework. These chords did not actually lift off the falsework until a vertical movement in the center of over 1 in. had taken place.

TABLE 2. VERTICAL MOVEMENTS, IN INCHES, OF NORTH ANCHOR ARM, QUEBEC BRIDGE

	Panel Points 12	10	8	6	4	2
Aug. 19	0	1/16	3/16	3/16	0	0
Aug. 24	1/16	3/16	3/16	3/16	1/16	0
Sept. 1	1/16	1/16	3/16	1/16	1/16	1/16
Sept. 3	1/16	3/16	1/16	3/16	1/16	0
Sept. 10	3/16	1/2	1/16	3/16	1/2	1/16
Sept. 15	3/16	1/2	1/16	7/16	1/16	1/16
Oct. 1	1/16	3/4	1 1/16	1 1/16	1	1/16
Oct. 8	3/8	1/2	1 1/16	1 1/16	1 1/16	1/16

The anchorage bars began to lift with the traveler at panel 4. As noted in a previous article (Jan. 7, 1915), the diagonal tension members and top-chord eye-bars of the anchor arm had been slotted in order to facilitate erection while the truss was in its cambered position. As the load was added to the cantilever arm, these tension members came to a bearing by degrees until the whole anchor arm was self-supporting.

When the erection traveler is removed, there will be a positive reaction at the anchor pier of about 200,000 lb.

*Assistant to Chief Engineer, Board of Engineers, Quebec Bridge, New Birks Building, Montreal.

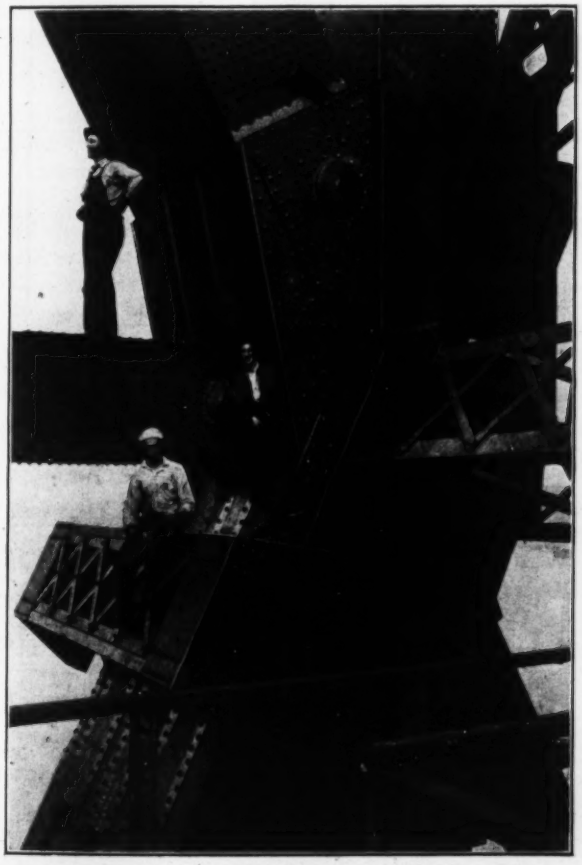
In order to put a tension in the anchor bars, the falsework bent under panel 2, next the anchor arm, will be left in and jacked up under a load of 600,000 lb. until the suspended span is swung.

The last work performed this season on the bridge was to erect and place the eye-bar suspenders for the suspended span, and to place in position the heavy jacking girders—on the extreme end of the cantilever arm—which will be used for supporting the jacks used in hoisting the suspended span after this is floated into position. As the eye-bars supporting the suspended span in place will be subject to constant movement, due to expansion and contraction of the bridge, the pinholes at the top and bottom sections have been fitted with phosphor-bronze bushings.

At the present time all falsework under the anchor arm has been removed, with the exception of the bent under panel 2, as mentioned above; and the erection traveler is being dismantled and taken down. This traveler will be used next season for the erection of the suspended span at Sillery Cove, some three miles down the river from the bridge, where falsework is being built close along shore as staging for construction of the span.

Progress of the South Cantilever

The inner falsework used on the north side to support the floor of the north anchor arm was last spring taken down and shipped to the south side and used in a similar position. As the shore line on this side is at a different elevation than on the north side, this difference in elevation was taken care of in the design of the falsework by leaving out or adding parts.



MIDDLE INTERSECTION OF FIRST MAIN DIAGONAL TO NORTH SHORE CANTILEVER ARM

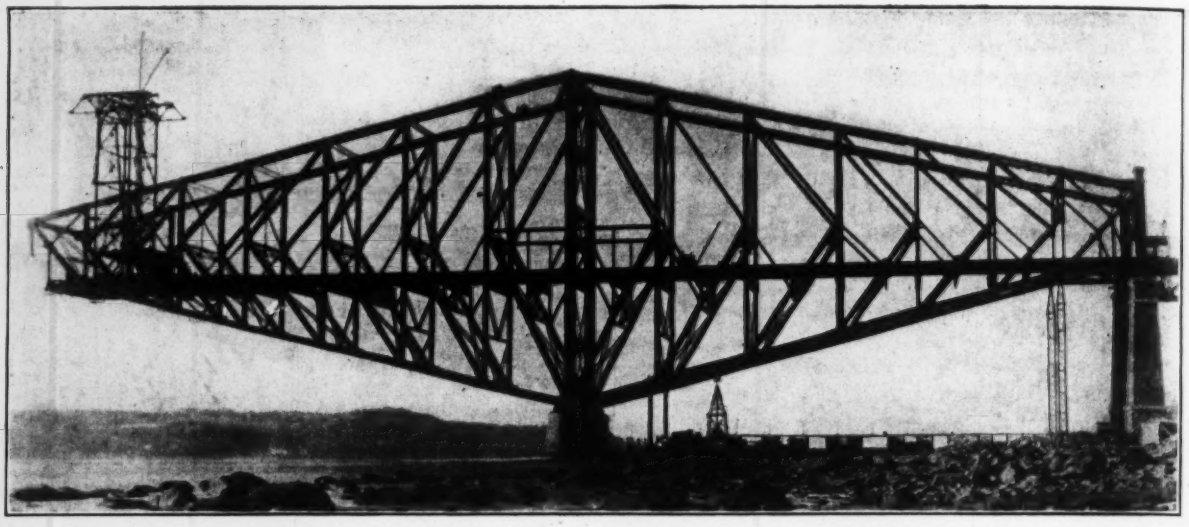
HOW THE STRAIGHT-LINE CANTILEVERS WILL APPEAR FROM RIVER (NORTH CANTILEVER, OCT. 25, 1915)

From the experience gained on the north side, very much better time was made on the erection of the south anchor arm, a saving of some six weeks being made. As on the north side, this work went together without any trouble, and the contractor was able to improve on the program laid out in the spring. Owing to the extra progress made on the south side this year, the contractor will have at least six weeks' advantage next spring and can start immediately on the erection of the cantilever arm. According to the present program, the south cantilever arm should be completed some time in September, 1916, at which time the bridge will be ready for coupling up the suspended span. As work up to now has gone ahead of program schedule, the bridge is likely to be ready for this operation at the time fixed.

Work on Suspended Span

The inside falsework under the floor of the south anchor arm has been removed and will be sent back to the shop to be remodeled for use as falsework for the erection of the suspended span. The concrete piers upon which this falsework will rest have already been put in place between low and high water at Sillery Cove, and early next spring a start will be made on the erection of the span.

The construction of the scows necessary for supporting the span during floating operations will also be started this winter. Six scows will be required, each 160x32 ft., with a maximum draft of 8 ft. under working conditions. The span, as floated into position, will be 640 ft. long, 88 ft. wide and 110 ft. high at the center, and will weigh approximately 4,700 tons. Steel links supported from the jacking girders at the end of the cantilever arms will be attached to the end of the suspended span, and with the falling tide the scows will be moved from under the bridge. During lifting operations all traffic will necessarily have

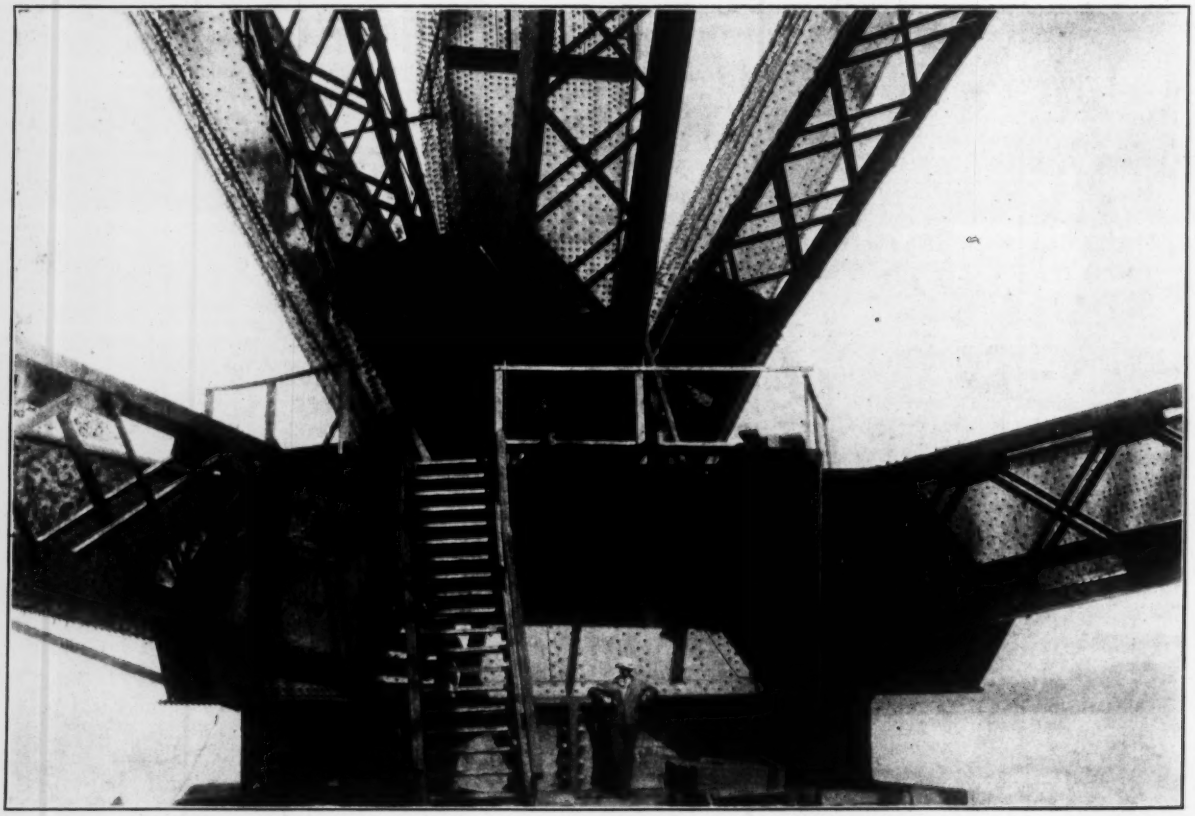


NORTH CANTILEVER OF THE QUEBEC BRIDGE ON DEC. 2, 1915

Falsework bent at panel-point 2 left in place to jack up for securing anchorage tension; erection traveler being dismantled

to be stopped in the river. If everything works according to the schedule, lifting operations should not take more than 24 hr. from the time the suspended span is hung to the end of the cantilever arm. By next fall it should be possible, after many years' effort, to have the two sides of the St. Lawrence River linked together at this point, finally carrying the great government railway, the National Transcontinental, across the St. Lawrence to its connections reaching the Atlantic coast.

The St. Lawrence Bridge Co., of Montreal, is the contractor for this work. Phelps Johnson is President; G. H. Duggan, Chief Engineer; George F. Porter, Engineer of Construction; W. B. Fortune, Superintendent of Erection; S. P. Mitchell, Consulting Engineer of Erection. The work is under the supervision of the Board of Engineers, Quebec Bridge, Department of Railways and Canals, Canada. The board consists of C. C. Schneider, Ralph Modjeski and C. N. Monsarrat, Chairman.



MAIN SHOE OF QUEBEC BRIDGE, WHERE TEN MEMBERS COME TOGETHER

Pick Rational Plant Units for Depreciation Accounting

By Robert G. Klotz*

The determination of what character of expenditure shall constitute in depreciation accounting a "renewal," and shall therefore be provided for in the depreciation account, and what shall constitute a "repair," and therefore be paid for out of expenses and regarded as maintenance, is a problem much encountered and the source of a great deal of misunderstanding between engineers and accountants, particularly in railway work. The application of a little commonsense will usually overcome the difficulties and result in a workable, if perhaps not a perfect, arrangement.

This question has been made a still more difficult one of late because of the detailed character of the appraisals prepared by engineers, voluntarily or to comply with the requirements of official bodies, bringing out the value of every possible detail. Without due regard to the true meaning of the actual depreciation in the physical condition of the property as shown in such reports and to the underlying purposes of depreciation accounting, accountants have in some cases attempted to employ the engineer's detailed valuation as a basis on which to form their depreciation accounts. Such procedure, while it may not result in any insurmountable difficulties, inevitably involves the accountant in a most intricate tangle of detailed accounts and in frequent misunderstandings with the engineer, whose purpose in accurately detailing his valuation was not to imply the need of writing up depreciation on each individual item entering into the construction of the property, though to the accountant it may so appear.

The real purpose of the depreciation account is to spread over the years of useful life the payment of heavy anticipated charges for the replacement of parts of the property when they can no longer be kept in operating condition by reasonable expenditures for repairs, where such charges, if met in one payment, would make the total expenditure for the year in which they are incurred disproportionately large.

Agree on Plant Units for Accounting

The selection of the proper units of plant upon which to base the charges should be made by agreement between the engineer and the accountant, or by some one person conversant with the requirements of both branches of the work. In this way the engineer can be certain that the depreciation fund will at the proper time be reasonably sufficient to cover the charges which will have to be made against it, and the accountant may be assured that he will not be required to carry an unnecessarily heavy burden of detailed accounts which might include items properly to be classed as repairs. In determining upon proper units there is ample opportunity for disagreement, and here the employment of sound commonsense and resort to reasonable compromise will be needed.

For true economy and for convenience the units used should be as large as is compatible with the equalization of the annual maintenance charges, and should be such units, or such parts of larger units, as it is fairly certain will have to be renewed as such.

In writing up depreciation on a generating station it would naturally occur to one that each generating unit, comprising prime mover, generator and appurtenances, might well be taken as the unit for depreciation, and an annual charge written up sufficient to provide for its replacement on the expiration of its estimated life. Here the engineer may, however, show that a certain appreciable portion of the unit may require replacement before the end of the assumed life; for example, that the insulation of the armature coils would be very likely to fail and that the coils would have to be replaced while the remainder of the unit was still in good operating condition.

Should it be necessary to renew the entire set of coils at one operation, an expense would be involved which might tend to increase unduly the expenditures for that particular year, in which case the armature coils or winding should be covered by a separate depreciation charge. Were it possible to replace individual coils from time to time, this would incidentally divide the unserviceable period of the unit into a number of short intervals rather than one long interval. Then the cost of the replacement might well be distributed over two or more years, in which case the expenditure in each year would not be sufficiently great to unbalance the annual maintenance charges, and the depreciation could be written up on the generating unit as a whole, instead of on two or more parts thereof.

Avoid Excessive Division of Plant

It is evident that whether the units assumed be large or small, the total cost of maintaining a unit, that is, annual repair and depreciation charges combined, should be the same. Where, however, the larger unit is used, as in the second situation cited, and it is possible to renew portions of a unit under the heading of repairs, depreciation is written up on each such portion only once during the life of the unit, although each portion may be replaced several times during that period. In this way excessive division of the depreciation account is avoided, and the account is at the same time kept at a minimum—a matter of some importance when it is considered that the amounts charged to the depreciation account must be regarded as practically lying idle.

Were it possible so to regulate the total annual cost of repairs and renewals in a plant that the sum involved would vary but slightly from year to year, or perhaps rise inversely with the decrease of value in some cases, a depreciation account would be obviously both unnecessary and undesirable; and if engineers and accountants will come together and discuss the question on a rational basis, they can in many cases closely approach such a desirable state of affairs. By extending the life of a certain unit and reducing the extent and cost of the repairs made during a certain year, with the knowledge that it will be removed from service during the year following, an amount may often be saved sufficient to provide for the renewal, covered by current funds, of another similar unit, the saving on repairs on the new unit during its first year of use providing sufficient leeway in the current funds for that year to cover the cost of replacing the unit previously allowed to deteriorate to the end of its usefulness.

To illustrate: If a certain length of track of a railway, all of which is subject to equal wear, has an assumed life of 20 yr., a sufficient amount may be charged to the depreciation account each year so that at the end of the 20 yr. the funds on hand will be sufficient to provide for its replacement, the cost of which will thereby be distributed

*111 West 49th St., New York City.